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Analysis of Dam Foundation in Alluvial Deposits, Method and Material Modelling

by

Mabin Dahal

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APPROVAL PAGE

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ABSTRACT

This thesis presents a study made to analyse a gravity dam foundation on alluvial deposits. Alluvial deposits consist of fine fraction and coarse fraction which can be characterised as boulder mixed soil. The percentage fines fraction of alluvial deposits has direct impact on the engineering characteristics of soil including density compactness and consequently affect the stability of Gravity dam in alluvial deposits. In this thesis, a method is presented to analyse the performance of gravity dam in alluvial deposits at natural and improved soil conditions. For this a relation is established between the percentage of fines fraction with the bulk modulus of elasticity, shear modulus of elasticity and density. This relation is used to predict the mechanical parameters after the improvement of soil with consolidation and compaction grouting which will transform the fine fraction into a cemented material. The value of void ratio, volumetric water content, permeability of improved soil is found out as a function of decrease in percentage fine fraction.

As on dam foundation the soil material is modelled as homogeneous and heterogenous material consisting different fraction of boulders and fines. FDM and FEM software are used to model the alluvial deposits at natural and improved soil conditions. Loading of dam is considered for full reservoir condition. The dam and foundation are assumed to be in plane strain condition.

The results from FDM and FEM analysis demonstrated the decrease deformation, porewater pressure and seepage in dam foundation with the decrease in percentage fine fraction in alluvial soil. The numerical analysis results showed that heterogenous material distribution better represent the ground condition. Seepage analysis results with and without cutoff wall shows the decrease in seepage with decrease in percentage fine fraction.

The dam foundation is analysed for vertical stress, vertical deformation, porewater pressure, and seepage.

Keywords: Alluvial deposits, Fine Fraction, Coarse Fraction, Gravity Dam, Displacement, Porewater pressure

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LIST OF ABBREVIATIONS AND SYMBOLS

F_V Vertical weight of dam

F_H Horizontal forces on dam

M_R Resisting moment

M_O Overturning moment

e eccentricity

U_1, U_2 Upthrust

k Permeability

K Bulk Modulus of Elasticity

G Shear Modulus of Elasticity

ρ Density

VWC Volumetric water content

η Porosity

c cohesion

1. INTRODUCTION

1.1. Introduction

Dam is a water retaining physical structure that blocks the flow of water. The primary purpose of dam construction is to collect and hold the flowing water for drinking water supply, irrigation, industrial use, recreational use, hydropower production, and flood control. During the period of excess flow, dam can be used to retain the water such that it can be released during the dry season. Dams are classified according to its size, height, function, structure and design. The various type of dams according to the construction material and method used are earthen, rockfill, hardfill, and gravity dam.

Alluvial deposit is a mixture of soil such as clay, silt, sand, gravel and boulder which has been eroded and carried by fast flowing streams and later settled when the velocity of flow decreases. The significant variations in thickness of the alluvial deposits and the presence of a low strength are the two primary problems to consider while constructing dam on alluvial deposits (Pawson and Rusell, 2014) (Hedayati Talouki et al., 2015). Gravity dam is constructed on hard strata and usually a bedrock is preferred whereas earthen dam is constructed on relatively weaker foundation. Due to heavy weight of gravity dam, it needs sound bearing capacity foundation to resist its load. Bedrock without geological discontinuities provide enough bearing capacity to construct gravity dam. There's always remain a possibility bed rock can't be found in river valley that is why dam has to be built within the alluvial soil layers. Performance analysis of dam foundation in alluvial soil deposits is necessary to predict its long-term stability. The alluvial foundations are vulnerable to the large settlements, which might results in undesirable settlements and even catastrophic failure (Jafarzadeh and Garakani, 2013). Thus, the foundation should be treated and performance should be analysed.

Rivers in Nepal passes through deep gorges which is full of alluvial, colluvial and sedimental deposits where bedrock lies much below the river surface. To construct a gravity dam is thus very challenging. RCC dam need a huge bearing capacity to withstand its enormous weight there remains huge risk of seepage due to waterhead and low binding properties of alluvial deposits. Seepage accelerates the internal erosion of soil where finer particles of alluvial deposits will be carried out by flowing water. This will result in the formation of cavities in an alluvial deposit. There are limited and variable procedure to analyse the performance of alluvial dam foundation. Thus, a dam study is necessary to analyse a way of simulating the performance of alluvial foundation.

This study presents a methodology to study and simulate the performance of alluvial dam foundation. The performance of dam foundation is analysed for natural and improved alluvial soil conditions. The effect of the mechanical parameters to withstand the gravity dam load in alluvial foundation is analysed. FDM and FEM software is used to analyse the behaviour of alluvial foundation at full reservoir conditions. The model incorporates the methodology to increase the strength of alluvial soil foundation from the combination of consolidation and compaction grouting. The results presented validated the methodology presented.

1.2. Background

Although we have a good recorded history of dam foundation all over the world. Until now, there is no defined way to simulate and analyse the performance of alluvial dam foundation to construct gravity dam on it. This might be due to its variation in engineering properties due to random heterogenous nature. "To pass judgment on the quality of a dam foundation is one of the most difficult and responsible tasks. It requires both careful consideration of the geological conditions and the capacity for evaluating the hydraulic importance of the geological facts." Karl Terzaghi, 1929.

Construction and design of dam on alluvial deposits is a difficult task due to the unpredictability nature of material below. Erosion of fine-graded soils through seepage can cause piping. It will result in differential settlement and seepage of dam. Moreover, alluvial deposits consist of mixtures of boulders, cobbles, gravels, pebbles, silts, coarse sand, fine sand and clay. The finer material between the boulder and cobbles can erode in the presence of high-gradient flow known as suffusion thus resulting the sinkholes in foundation of dam and abutment.

But in recent times, at clear lake dam a concrete dam was constructed above alluvium deposits of cobble, boulder, gravel, sand, silt, clay with proper design of filter materials, seepage control mechanism, grouting materials (Buczek et al., 2018). At Gerdebin dam, Iran there is weak and soft alluvial deposits beneath it and rockfill dam was constructed beneath it (Jafarzadeh and Garakani, 2013). Tamakoshi dam of Nepal is also built in alluvial deposits. There is a 100m thick deposits of alluvium and colluviums material in Tamakoshi hydropower dam It is very difficult to reach bed rock and make a dam.

Deposition of materials are arranged in a way that boulder remains at bottom of river then coarse material remains above it and consequently fines material stays top of it. If we construct a gravity dam above it, there is a chance that finer material will squeeze, and boulder material will crush which will ultimately fail the dam.

A research was conducted on “Clear lake dam” having heterogenous deposits of boulders, cobbles, rockfall, alluvium and lacustrine deposits proposed a solution to minimize cracking, control seepage, and bearing capacity on existing condition (Buczek et al., 2018). The primary difficulties in the design was supporting dam on the weak alluvial deposits, controlling seepage gradient, and proving proper filter and drain system at different portion of dam. A roller compacted dam with proper control of seepage gradient, filter drainage system, cutoff wall is able to withstand in weak foundation. Another case study was done at Chapar-Abad dam (Uromeihy and Barzegari, 2007) which have alluvial deposits of over 60m thickness as base material and having various coefficient of permeability. Probable permeability and seepage on foundation was estimated by conducting in-situ tests, and by numerical modelling. Based on those results installation of grout curtain is suggested to reduce seepage.

The primary factors which governs the selection of treatment method in dam foundation are seepage, hydraulic gradient, safety factor and cost. According to the soil types different methods like excavation of alluvium (permeable material), clay blanket, stone columns, jet grouting, deep mixing, compaction grouting, consolidation grouting, grout curtain etc. can be applied to improve the seepage and enhance the bearing capacity of alluvial deposits.

Foundation of dam going to bed rock in alluvial deposits is extremely difficult and not cost effective especially for small low-budget hydropower projects. Sometimes the bed rock also can’t be found. No such case study has been done yet on construction of dam in alluvium deposit by taking the Nepal scenario. This may be due to the fact that major hydropower structure designs in Nepal are given to foreign consultant. Analysing behaviour of dam foundation from view point of bearing capacity and seepage in an alluvium deposits at natural and modified conditions is necessary. It helps to know the long-term performance of dam as well as it eliminates the chances of sudden catastrophic failure. Therefore, a method is necessary to simulate the performance of alluvial deposits to dam load at natural and improved soil conditions.

1.3. Objective

- Study and review the dam foundation in alluvial deposits.
- Derive a methodology to study and simulate the performance of alluvial dam foundation.
- Analyse the stability of dam foundation in alluvial deposits with and without improvement.

1.3.1. Secondary Objective

- Derive a methodology to model the improved soil material via grouting.
- Derive soil parameters for improved soil.
- To analyse the alluvial dam foundation with improving alluvial deposits.
- Analyse the foundation seepage before and after soil improvement.

1.4. Scope

The analysis of dam foundation in alluvial deposits is performed at natural and improved soil conditions. Boreholes data and geotechnical survey report of multi-channel analysis of the surface waves (MASW) report is collected, analysed and a relation is found out which gives percentage fine fraction variation with moduli of elasticity and density. This relation can be used to find out the value of mechanical parameters in alluvial deposits. The report of field permeability test is collected and analysed to find out value of permeability of soil at natural conditions. Values of permeability, void ratio, porosity and volumetric water content is found out at natural and improved soil deposits as a function of grain size distribution.

To represent the actual field condition linear distribution and heterogenous distribution of material is done at natural and improved soil conditions. Performance of gravity dam is analysed in alluvial deposits from the view point of porewater pressure, bearing capacity and seepage. While assuring gravity dam regarding bearing capacity on alluvial deposits, finite difference software is used.

The seepage analysis is conducted on GeoStudio with and without cutoff wall for natural and improved soil conditions. The change in value of seepage with and without the provision of cutoff wall is observed and analysed.

1.5. Methodology

This thesis presents one of the ways to analyse gravity dam on alluvial deposits. The method is presented to analyse the performance of gravity dam in alluvial deposits at natural and improved soil conditions. The methodology chapter presented the analysis procedure and parameter development for natural and improved soil conditions. The stiffness of alluvial deposits is increased by combination of compaction and consolidation grouting. The assumption is that the fine voids in the alluvial matrix filled with grouting to improve their performance. With combination of different types of grouting the overall stiffness of foundation material increases. This means overall percentage fine fraction of foundation material decreases. The relation between percentage fine fraction with moduli of elasticity and density is find out on alluvial deposit. For each decrease in percentage fine fraction, dynamic properties of soil and density is found out using the calculated relationship between percentage fine fraction with mechanical parameters. The material modelling of homogenous and heterogenous material is performed. The performance of foundation material is checked considering linear homogenous material distribution and heterogenous material distribution. Pore pressure, settlement and foundation pressure are investigated for natural and improved soil. Further, GeoStudio is used to calculate the seepage using the value of permeability for decrease in percentage fine fraction. Seepage is calculated with and without the provision of cutoff wall for natural and improved soil.

1.6. Content of this thesis

The chapter first is introduction and it introduced overall thesis works. The various problem in alluvial dam foundation and its treatment method is described. Chapter two is literature review which elaborated the dam and its foundation. It also explains the five case histories of dam foundation on weaker deposits. In chapter three a method is presented to analyse the performance of dam foundation in alluvial deposits at natural and improved soil conditions. Different parameters required for analysis are developed for natural and improved soil conditions. In chapter four result and discussion, FDM and FEM software is used to validate the method presented from the view point of seepage, porewater pressure, vertical stress and deformation. In chapter five, a conclusion of study is made. In chapter six and seven limitations and recommendation of study is discussed.

2. LITERATURE REVIEW

2.1. Introduction

A dam is a water retaining structure that stops the flow of surface and subsurface water. The water retained by dam are used for activities like irrigation, sanitation, human and animal consumption, recreational activities, protection of animal habitat.

2.1.1. Classification of dam

A. According to the functions of dam, it is categorized as follows:

1. Storage dams: These types of dam are built to store water during monsoon season where there is abundant of water and later release during the dry summers. They are usually much needed for hydropower during dry seasons. It can also be used for recreation, fishing, drinking water for wildlife.
2. Diversion dam: Diversion dam are built for diverting purpose of river into an off-taking canal. They are small storage which are used for irrigation and diverting stream into storage reservoir.
3. Detention dams: The main purpose of detention dam is to control flood. It prevents the flow of river in downstream during flooding which helps to protect the area. The collected water is later released in a control quantity.
4. Debris dams: Debris dams are designed and built to block the boulder, cobble, gravel, sand and various other materials flowing with water. The water which pass through debris dam become clear to some extent.
5. Coffer dams: It is a boundary constructed around the construction site so that construction work can be done inside it in dry condition. They are temporary structure which are usually made to construct structure in river, pond, sea, lake etc.

B. According to structure and design, dams can be classified as follows:

1. Gravity Dams: A gravity dam is a massive sized dam made from reinforced concrete or stone masonry. They can usually hold back large amount of water. By using its own weight, it can control horizontal forces exerted by retaining water. Since it depends on its own weight so it has to be built on solid bedrock. They are suitable for stopping water in narrow gorge and wide valleys.

2. Earth Dams: A dam which is made of earthen material, constructed by compacting different layers of earth. The impervious material is placed on core whereas pervious material is made on upstream and downstream sides. To prevent dam from erosion crushed stone is used. It resists forces exerted upon it due to shear strength of soil. They are usually built on weak foundation like alluvial, colluvial, lacustrine deposits. It can be built on all types of foundations. the height of earthen dam directly depends upon foundation strength.
3. Rockfill Dams: A rockfill dam is constructed from boulders of large size and fragmentation of rock. An impervious membrane made of concrete or asphalt is used as an impervious membrane in upstream part of dam. A dry rubble cushion is put inside the rockfill and membrane to distribute the water load and support membrane. They are usually done in area where large quantity of rock is available in nearby areas.

2.2. History of Dam Construction in context of Nepal

The power system in Nepal is heavily dominated by hydropower system. Nepal has 600MW of installed capacity in its Integrated Nepal Power System (INPS) where about 90% of countries power system is generated from hydropower. Nepal first hydropower was constructed in 1911 at Pharping which produced power of 500 KW.

Nepal has huge potential on hydropower due to higher Himalayas and ever flowing water. Although the theoretical power potential is believed to be in the range of 83,000 MW but considering economic feasibility its potential is tentatively 43,000 MW. First hydropower plant of 500KW(Sharma and Awal, 2013) was established in Pharping then the second of 640KW capacity was developed in Sundarijal in 1936. In 1942 a Sikarbus Hydroplant of 640KW was built on Chisang Khola by Morang hydropower plant. Later, the hydropower plant was demolished during 1960s landslide. Until 1962, the Electricity Department of HMG has authority to generate, transmit, and distribute electricity. Nepal Electricity Corporation (NEC) was formed in 1962 and given the responsibility to transmit and distribute electricity but electricity department was given responsibility to generate electricity. Then later, Panauti Hydroplant of 2400KW was established in 1965 subsequently Trishuli Hydroplant of 21000 KW was built in 1967. The Eastern Electricity Corporation was established in 1974 then small hydropower development board was established in 1977. In 1985 after the collaboration of electricity department, Nepal Electricity Corporation and all the developmental boards excluding the Marshyangdi

Hydropower Development Board, the Nepal Electricity Authority (NEA) was established. After this collaboration, NEA has the authority for the generation, transmission and distribution of electricity. Water and Energy Commission and its Secretariat established in 1976 and the policymaking body formed in 1981 and Department of Electricity Development are the few public sectors which are attached in hydropower production. Lately, private sectors are also performing a vital role in hydropower development. Until 2005, the total hydropower production in Nepal is 556.8 MW which is 0.7 percent of its potential.

The first hydropower plant of Nepal is Pharping Hydropower Plant which has power generation capacity of 500-KW. It is also considered as oldest hydropower plants in Asia. It was consecrated on May 1911.

2.3. Dam and its components

A dam is a water retaining structure that stops the flow of surface and subsurface water. The water retained by dam are used for activities like irrigation, sanitation, human and animal consumption, recreational activities, protection of animal habitat. Above all these it is mostly used in production of hydropower. They can also be used to control the impacts of floodwaters, enhance river navigation and should be operated in a way that it should increase downstream water quality.

2.3.1. Components of Earthfill dams with function

The various components of Earthfill Dams with their function are:

1. Core

   The primary function of core is to enhance the water tightness of dam. It is made from impervious material where size of the core material is very fine so there won't be seepage problem in it.

2. Shell

   Shell is built up of porous material which primary function is to provide strength and support to the core wall where coarse material are uses.

3. Transition filters:

   The purpose of filter in dam is to stop mixing of finer material from core and coarser material from shell. It is usually semi-pervious in nature.

4. Cut off trench:

   The purpose of Cut off trench is preventing seepage and is made up of impervious material.

5. Sheet pile wall:

   Sheet pile creates a boundary wall which stops the water flowing across it. When there is the presence of pervious layer across soil to prevent the water entry sheet pile is constructed.

6. Impervious stream blanket:

   To prevent the seepage in dam foundation impervious stream blanket is laid out.

7. Rip rap

   The primary function of rip rap is to prevent upstream face from erosion. Its size is from 0.5m to 1m and usually made up of boulder.

8. Soil turfing/Sod:

   The primary purpose of soil turfing is to prevent erosion on dam due to rain, snow fall, wind etc.

9. Crest/top:

   Crest is upper part of the dam which divides the upstream face and the downstream face. Its primary objective is to access movement over it.

10. Free board:

    The function of free board is to protect dam from overtopping.

2.3.2. Components of concrete gravity dam

The various components of Concrete Gravity dam are:

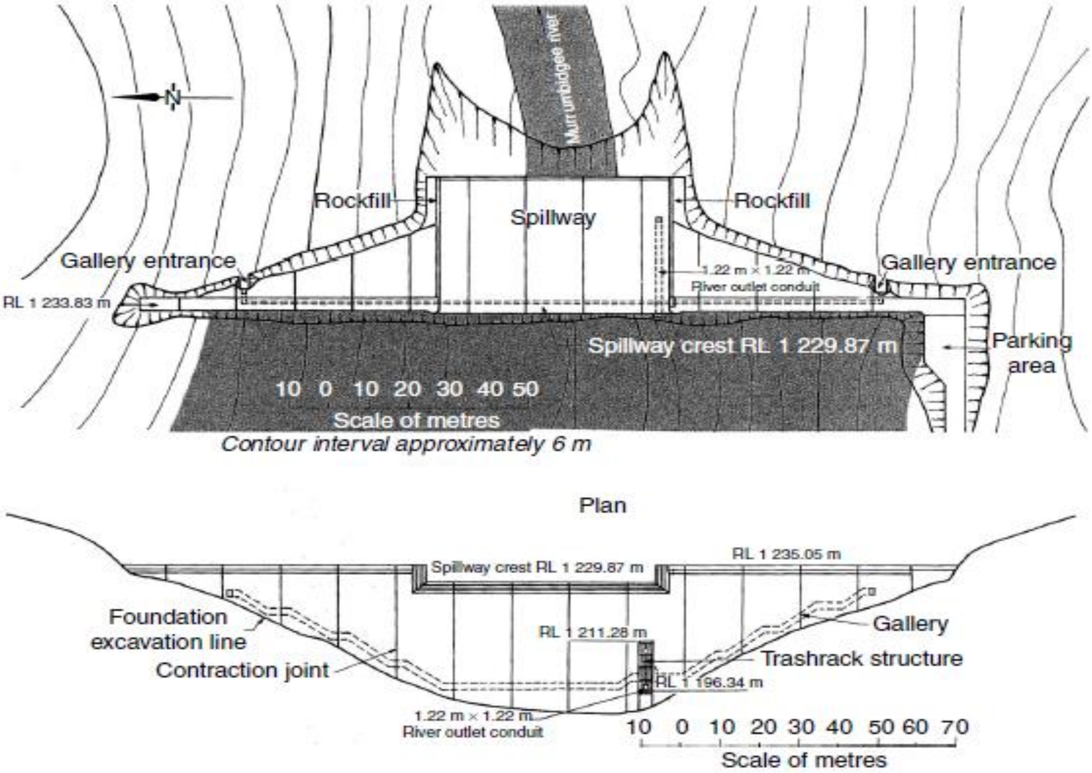

Figure 2. 1: Typical concrete gravity dam plan and elevation (Fell et al., 2014)

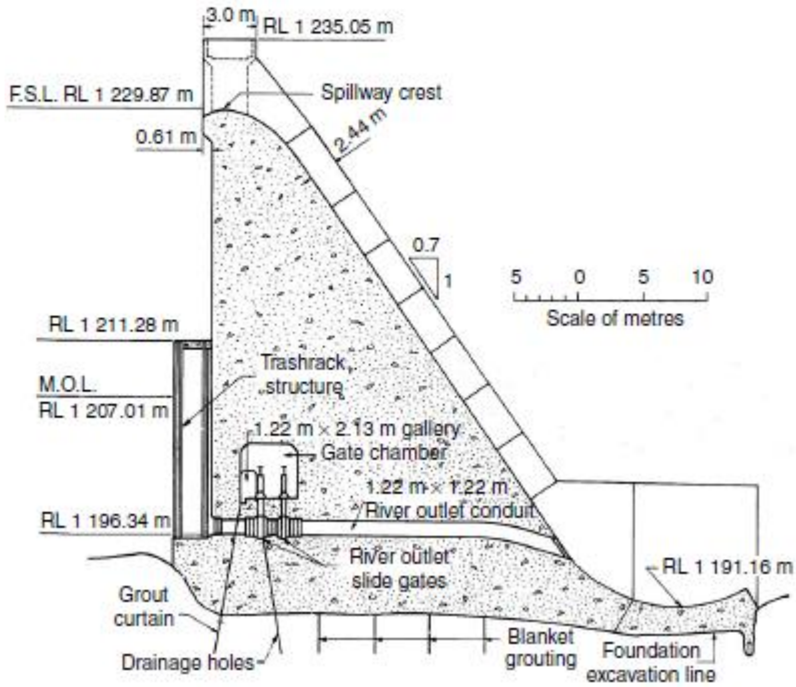

Figure 2. 2: Typical concrete gravity dam cross section (Fell et al., 2014)

2.4 Dam foundations

Concrete dam is usually built on rock strata due to its heavy weight. Rock foundation is always sound in terms of bearing capacity, resisting erosion, and reducing seepage. There is no height limit on RCC dams when constructing on sound bedrock so, it has been constructed at heights up to 156m. While constructing dam on rock, engineers always try to avoid the area having fault, fold, bedding plane, shear zone, discontinuities because addressing these problems cannot be cheap. Proper investigation and research are always necessary and is important in foundation design and construction of dam than its section.

While constructing dam on soil engineer's must determines its compressive strength, shear strength, deformation modulus, Poisson's ratio, permeability, hydraulic conductivity. It helps to find out the bearing capacity of soil material. Compressive strength of soil governs the thickness of dam base to distribute its weight efficiently. The friction between the particle of foundation rock determines the shear strength of its own. Usually dam foundation is built on area that do not have total change in deformation modulus across the dam foundation. The complete changes in deformation modulus across the dam foundation may results in differential settlement and cracking. For more critical analysis of the foundation transverse and axial strain of soil must be determined which ratios is also called Poisson's ratio. The coefficient of permeability must be determined at different cross section and layers of dam cross section. Based on the value of permeability and hydraulic conductivity of soil improvement measures is applied.

2.4.1. Foundation geology and geological structure

1. Solid rock foundations:
    - They have enormous bearing capacity and are suitable for all types of dams.
    - They are usually homogenous and impermeable.
    - The rock which are highly fractured should be removed and cracks can be filled up by injection of grout.
2. Gravel foundation:
    - Both the load bearing capacity and permeability of gravel foundation is high.
    - Gravel foundation are unfavourable for arches and splintered dams.
    - If highly compacted, it is favourable for rock filled, soil filled and gravity dams.

3. Silt or fine sand foundation

- In silt or fine sand foundation bases, there occurs large settlements and high permeability due to low strength.
- These types of foundation are appropriate for low concrete and soil filled dams.
- The major problems in this type of foundation is excessive infiltration and basic settlement.

4. Clay foundation

- Their load bearing capacity is minimum.
- Because of consolidation, permeability is low on clay foundation
- This type of foundation is appropriate for low-earth fill dams.
- In this scenario, experienced designer and project supervisor is needed.

2.5. Factors affecting selection of dam site

During planning phase input from geotechnical engineers, structural engineers, surveyors, geologists, hydrologists, ecologists etc. is required. Before choosing site, designers must conduct a comparative study regarding design with focusing on its primary objective which can be hydropower electricity generation, water supply, flood control measures. When one of the alternatives is choose the following careful thoughts should enters in the design and construction of dam. They are:

- Hydrological data of climate and streamflow.
- Geological and geotechnical data regarding foundation design.
- Environmental impact of the dam impoundment.
- Material selection and construction techniques.
- Proper methods for rerouting of river during construction.
- Estimation of sediments to deposit in future.
- Analysis of dam safety regarding bearing capacity, seepage, stability

During the operation phase of dam water is released through gates and for generating maximum electricity hydraulic head is always maintained. There is always an interest of conflict for river system having many reservoirs, for dams used for various purposes, and when there are numerous social, economic and environmental impacts.

2.6. Requirements of site investigation

The Site Investigations must include data collection to provide information on:

- Geological history of the foundation area,
- Stratigraphy,
- Tectonic geology,
- Faulting, Foliation and Jointing of the foundation rocks,
- Structural relationships between the rock units,
- Permeability of the rocks,
- Seasonal groundwater variations and chemical composition of the groundwater,
- Deformation characteristics and strength characteristics of the materials in the stressed zone below the large composite dam foundation.

2.7. Detailed Investigation at Dam Site

The detailed study report at dam site should include following information:

1. Topographic surveying
2. Geological mapping
3. Underground exploration ex: seismic refraction, Boreholes
4. Detailed hydrogeological studies
5. Slope stability analysis at different cases

2.8. Factors affecting dam type selection

The two geological factors which affects the dam type selection are topography and geology. The geotechnical factors which governs the selection are bearing capacity of the underlying soil, settlement, and permeability. Also, material availability, spillway position, earthquakes, preferred safety, height of dam, aesthetic view, labour experience and cost affects the dam type selection.

2.9. Factors Governing the Place of the Dam Axis

The various geological, geotechnical, hydrological, social and economic factors which governs the place of the position of dam axis are topography, overall geology of site, underground materials, spillway location availability, sediments in the flowing water, water quantity on wet and dry season, expropriation costs, Seismic vulnerability, downstream water rights.

2.10. Dam construction

There are primarily four types of dams according to its method of construction. They are arch, buttress, gravity, and embankment dams. The construction of dam depends upon intended use of the structure, location, water volume to be retained, materials available, and budget.

2.11. Forces Affecting to Dam

A dam should be strong enough to resist all the static and possible dynamic forces which acts on it. The most important forces to consider in dam are as follows:

- Self-weight of dam
- Hydrostatic force
- Uplift pressure and porewater pressure
- Seismic force
- Freezing water pressure

2.12. Requirements of dam foundation criteria

2.12.1. Seepage Control Criteria

There is various method to control seepage in dam foundations which are construction of horizontal drains, cut-offs, placement of upstream impervious blanket, provision of downstream seepage berms, relief wells, and trench drain(Han Jie, 2015). Before selecting an appropriate method for seepage control the merits and effectiveness of various available methods should be analysed by using flow nets or FDM or any other numerical method. Especially for weak foundation the changes in seepage, uplift pressures at different points should be calculated.

1. Horizontal drains

Horizontal drain noticeably decreases the uplift pressure in the foundation. It increases the seepage quantity beneath the downstream of dam.

2. Cut-offs

In case of alluvial soil deposits, geotechnical engineer should decide total cut-off or allowance of limited under seepage in a control way. Complete cut-off includes concrete wall, compacted backfill trench or slurry trench. And partial cut-off includes upstream impervious blanket, downstream seepage berm, toe trench drain, or relief walls.

The following factors are the reasons to choose under seepage control measure (Sherard 1968):

1. Economic value of seepage loss vs the cost of total cut-off.
2. Foundation vulnerability to piping
3. If the water pond exists at the downstream of dam due to seepage in that scenario complete seepage cut-off is more desirable.
4. The quantity of silt and clay deposit in river which contributes in the siltation of reservoir with time, tends to reduce seepage.

a. Compacted Backfill Trench

It is an effective method of controlling seepage in which we excavate a trench below the impervious zone passing through pervious zone. And, we refill it with compacted impervious material which also provide full-scale exploration trench. The seepage gradient is maximum at base of cut-off and on downstream face in foundation and embankment zones where proper filter should design to resist piping.

b. Slurry Trench

When the thickness of pervious foundation is more in that scenario compacted backfill trench can be expensive. Therefore, in that scenario slurry trench can be a good option to control underseepage. During the construction of slurry trench, a trench is drilled to support the drilling hole a bentonite clay is used. The trench is then filled with finer materials passing through the No. 200 sieve to make it impervious at the same time with containing coarse particle to control settlement.

c. Concrete Wall

In case of alluvial deposits containing heterogenous mixture of boulders, cobbles, gravels, sands, silts, clay particles etc. or foundation exceeding the thickness of pervious material more than 45m then in that scenario concrete cutoff wall is very good option. In this method a concrete wall is made in a bentonite supported trench.

d. Steel sheet piling

This is not usually use as a cutoff wall to block seepage under the dam due to low head efficiency. They are usually use as a boundary structure for soil confinement to prevent soil form piping.

2.12.2 Deformation control criteria

Bearing capacity of foundation determines the load carrying capacity of dam. Without adequate bearing capacity deformation occurs on the foundation. Enhancing the bearing capacity of foundation requires seepage control measures, deformation control measures, and seepage gradient control measures.

Stiffness or modulus of elasticity of dam foundation material is necessary to find out possible deformation during loading. Depending upon the heterogeneity and material composition the possible deformation for particular loading is determine. After calculating the possible deformation, particular ground improvement technique or combination of more technique is select. The techniques range from replacing the foundation material to enhancing its soil properties. In case of lacustrine deposits there is no option than replacing it with selected fill. In case of alluvial deposits combination of compaction and consolidation grouting is necessary to enhance bearing capacity. More the percentage of finer material in deposits, more the treatment is necessary.

To evaluate the quantity of grout essential in ground improvement, sensitivity analysis is requisite. In sensitivity analysis comparison of deformation is mandatory from no grouting condition to pure concrete zone. This process gives us optimum quantity of grouting.

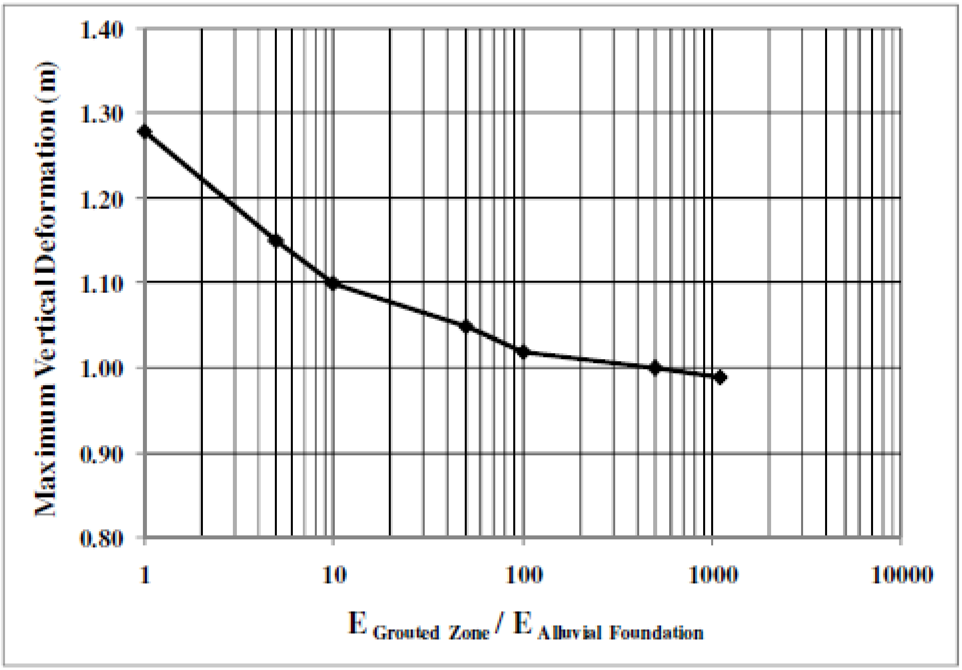

Figure 2. 3: Maximum vertical deformation versus different grouting stiffness (Jafarzadeh and Garakani, 2013)

To check the effectiveness of the grouting, it is necessary to compares the vertical deformation, lateral deformation, vertical total stresses on two occasions with grouting and no grouting conditions.

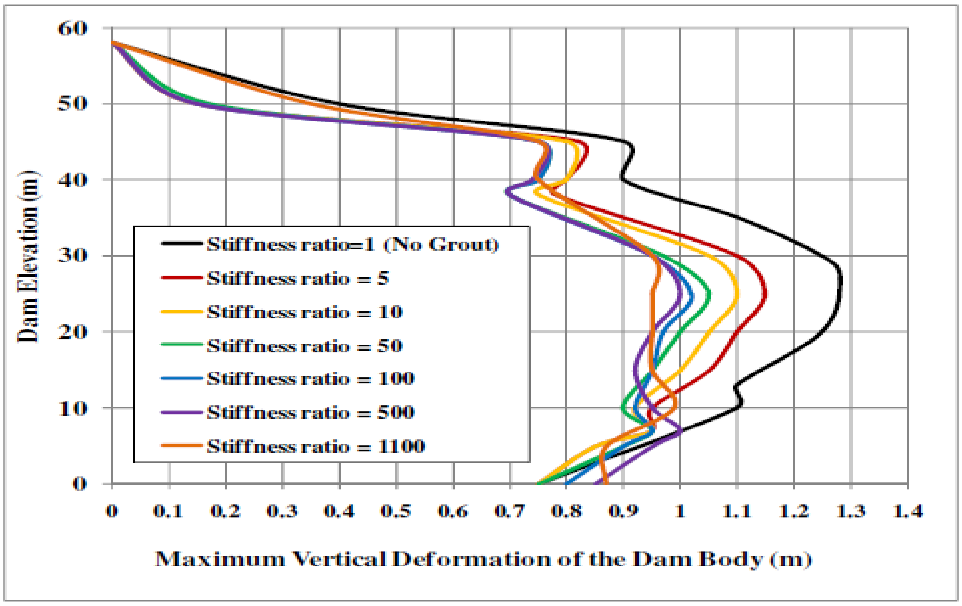

Figure 2. 4: Vertical deformation versus dam elevation for different grouting properties (Jafarzadeh and Garakani, 2013)

The plots of vertical deformation versus dam elevation for different grouting properties is necessary. It helps to find out the depth of maximum vertical deformation for particular grouting ratio.

2.13. Properties of alluvial soils

Alluvial soil is loose soil or sediments which consists of mixture of clay, silt, sand, gravel, cobbles, boulders etc. which have been eroded by fast flowing streams and carried in suspension by flood or river water before being settled at low flow velocity. This is a continuous process on river. The thickness of deposition also increases on each passing year which ultimately rises the river flow level. There will be a large thickness of sediment below river flow level. In large thickness of alluvium soil, there will be presence of many layer, and each layer will have different coefficient of permeability. Alluvial soil deposits are always vulnerable to larger differential settlement ultimately resulting into catastrophic failure of structures resting above it(Fell et al., 2014). Thus, the dam which has to be built above it should be given proper consideration towards seepage and bearing capacity.

Alluvial deposits are usually deposited in the channels and flood-plains of rivers and in lakes, estuaries and deltas. Due to the random heterogenous nature of it, it has huge variation in characteristics and are usually anisotropic in nature. Its characteristics ranges from clays of high plasticity to coarse sands, gravels and boulders. The alluvium material is usually unconsolidated, i.e. not formed together into solid rock, and can be eroded and carried away by flowing water before being settled somewhere else when the velocity of water decreases.

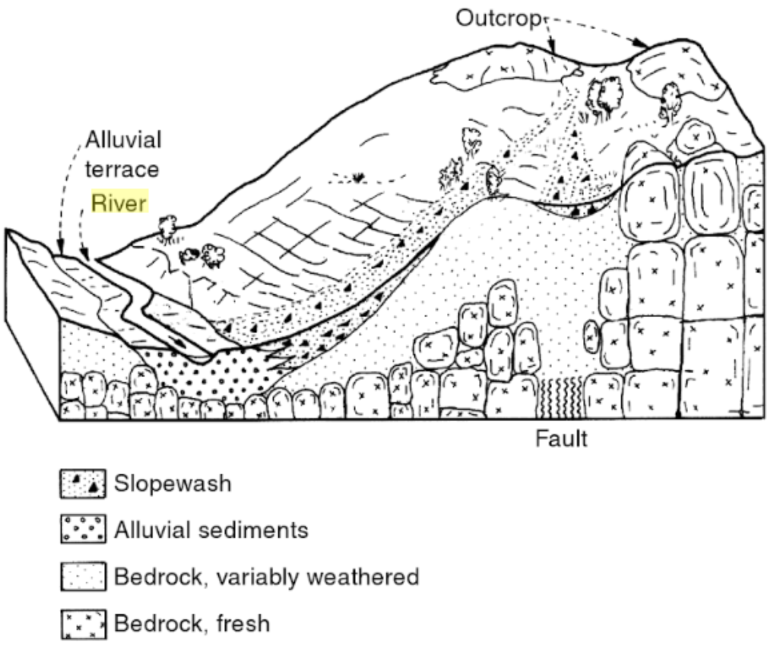

Figure 2. 5: Systematic view of residual, slope wash and alluvial deposits (Fell et al., 2014)

2.14. Properties of colluvial soils

Colluvial soils include the soil deposits which have been eroded and move away by gravity forces in presence of water and usually deposited when velocity decreased. It ranges from clays having high plasticity to boulder talus deposits. In landslide colluvium there is heterogenous mixture of clays gravel and boulder. They vary their properties with each deposit.

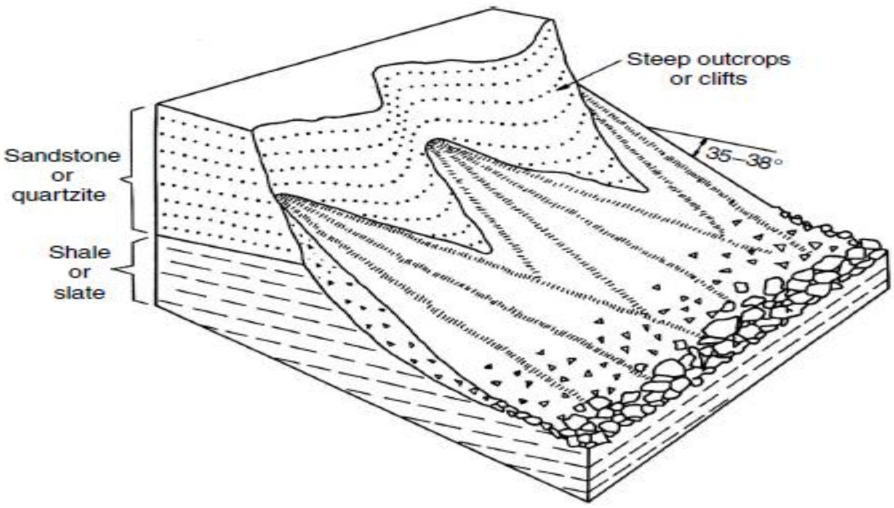

Figure 2. 6: Schematic view of residual, slopewash and alluvial deposits (Fell et al., 2014)

2.15. Curtain Grouting

Grout curtain are thin, vertical, cylindrical grout walls which act as a water barrier in a foundation. They are made directly into the soil by injecting grout on certain pressure at closely spaced intervals. They are constructed in a certain defined spacing according to its design. Spacing is design in such a way that each column of grout intersects the next. Then, there will be formation of a continuous wall or curtain(Weaver and Bruce, 2013).

2.16. Cutoff Walls

Cutoff walls is a way of controlling seepage through dam and its foundation. They alone cannot be assured totally effective because of inability to see subsurface during their construction. They are often used along the grout curtain and designed filter elements.

Cutoff wall are used to reduce seepage by eliminating seepage energy. They are mostly used on upstream part of dam centreline where pore water pressure and seepage gradients are much less likely to have less adverse effect on all-around performance of dam(Weaver and Bruce, 2013). It helps to protect dam from seepage. It is usually made during initial construction or during repair. It helps to increase the foundation strength.

The primary factors in selecting a treatment method in dam foundation are seepage, hydraulic gradient, safety factor and cost. According to the soil types different methods like excavation of alluvium (permeable material), clay blanket, stone columns, jet grouting, deep mixing, compaction grouting, consolidation grouting, grout curtain etc.

can be applied to improve the seepage and enhance the bearing capacity of alluvial deposits.

2.17. Seepage and Porewater Pressure

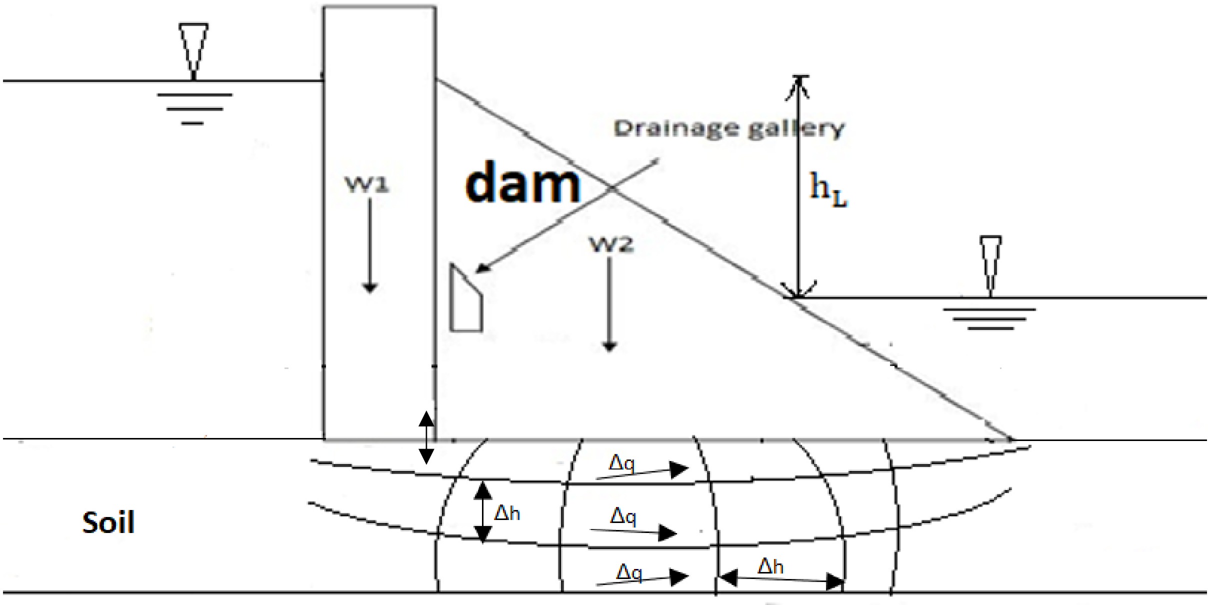

Figure 2. 7: Seepage Beneath Concrete Gravity Dam

The term streamlines represent the flow path of water molecule in flow region as shown in Figure 2.8 There are thousands of streamlines in the flow region. The passage between two adjacent streamlines is known as flow channel. An equipotential line is a contour of constant total head and they are drawn in such a way that the total head difference between two adjacent ones is same.

The movement of water from upstream to downstream through the soil as shown in figure causes the loss of some amount of energy head known as the head loss h_L which is equal to the difference of water level and upstream and downstream of the dam. The pressure of water inside soil at any given point in the flow region can be determined as:

$$\mathrm{U} = \text{pressure head x } \rho_w \text{ x g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; Equation\ 2.1$$

Where,

U = pore water pressure

ρ_W = the density of water.

Considering downstream water level as datum, it makes total head within downstream as 0 and upstream as h_L respectively. Head loss of $h_L$occurs along each streamline originating upstream and ending at downstream.

The total seepage through the soil, can be given by:

$$q = k\, h_L \frac{N_f}{N_d} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; Equation\ 2.2$$

where,

k = soil permeability

h_L = total head loss

N_f = Number of flow line

N_d = Number of equipotential drops

2.18. Dam build in alluvial foundation

The success of RCC dam on Clear lake dam (Monley et al., 2018) enhances the possibilities of construction of gravity dam on alluvial deposits of boulders, cobbles, gravels, sands, silts, clays. The detail geological and geotechnical study of foundation material below the upstream portion, dam portion and downstream portion of the dam is a must. It helps to find out the exact soil properties beneath the dam and their stress-strain behaviour. The various possible problems in dam foundations are seepage, hydraulic gradient and deformation. These three problems are interconnected. Occurrence of seepage triggers piping in a deposit which ultimately helps in settlement. After piping segregation of fine particle occurs, seepage gradient increases. The various type of solution to those problems is cut-off wall, clay blanket, grouting, grout curtain, soil replacement (Uromeihy and Barzegari, 2007) (HOSSEIN et al., 2015). The solution depends upon the constituent of boulders, cobbles and other fine particles in a deposit.

Case I: Clear Lake Dam Replacement: RCC Dam on a Challenging Soil Foundation (Monley et al., 2018)

Clear Lake Dam have a varying heterogenous mixture of boulders, cobbles, rockfall, alluvium and lacustrine deposits. The depth of deposits is more than 30 meters. These types of deposits have low strength layer and low bearing capacity.

The foundation material is soft, weak, and have different layers of it. Each layer has various degrees of permeability in it. Seepage and seepage gradient are found to be the primary sources of all problems. There might be the various causes of seepage depending upon the soil type condition. The primary cause of seepage in Clear Lake dam is found to be suffusion and erosion along outlet conduit in the foundation soil.

The post monitoring after construction of RCC dam on Clear lake dam illustrates that it is possible to construct RCC dam on alluvial deposits of boulders, cobbles, gravels, sands, silts, clays etc. The geological and geotechnical properties of foundation beneath the upstream portion, dam portion and downstream portion of the dam is necessary to study in details. It helps to find out the exact soil properties beneath the dam and their stress-strain behaviour. Seepage gradient in the upstream portion of dam can be minimise through the use of cutoff wall below the abutments. Below the dam portion seepage gradient and piping must be control. This can be control through the design of filter and cutoff wall. Below the downstream portion of dam, exiting seepage and seepage gradient needs to be find out. Seepage value determines the diameter of drainage pipe for safely discharge of water. Similarly, seepage gradient determines the possibility of suffusion. Through the value of seepage gradient, we can determine the effectiveness of our ground improvement techniques.

Case II: Foundation Treatment of Embankment Dams with Combination of Consolidation and Compaction Grouting (Jafarzadeh and Garakani, 2013)

Gerdebin dam consists of soft and weak alluvial deposit. The thickness of deposit is 48 metre. It is made up of various layers. Due to this it has low bearing capacity and highest seepage rate. Rock fill dam is construct in it. Rock fill dam is heavy and there remain chances of deformation in the foundation soil. It is very much necessary to control deformation in Gerdebin Dam. Deformation in Gerdebin Dam is control by combination of consolidation and compaction grouting. Grouting increases the stiffness of the foundation material. The effect of combination of compaction and consolidation grouting is performed by using finite difference-based software. The effect of changes in mechanical properties of foundation material helps to determine the increase in bearing capacity. At certain mechanical properties the dam body reaches the minimum deformation. The mechanical properties are dependent on grouting properties and its pattern of implementation. Optimization of grouting pattern helps to determine is achieved.

There is a presence of weak alluvial deposit in the foundation. Above it there is a heavy weight rockfill dam. Foundation consists of low strength due to which there is a possibility of differential settlement. There is also a chance of catastrophic failure of dam. To protect the dam from vertical deformation is the prime objective of this case study.

In soft soil deposits, due to its low strength layers there is always a possibility of deformation after the construction of structure above it. Treatment of soil structure is necessary to withstand dam over it. The degree of treatment and types of grouting depends upon the material properties of dam foundation and estimated load of dam which rests above it. The vertical deformation of dam foundation needs to be found out for no grouting to different grouting ratios. The depth of maximum vertical deformation for different grouting ratios along with measurement of maximum vertical deformation is necessary. The comparison of these grouting ratio and their deformation value helps to find optimum grouting ratio. This optimum value will make the deformation control measures cost effective. This is calculated by using finite difference software.

Case III: Skhalta Dam -Design of a hardfill dam founded on deep alluvium and lacustrine deposits (Pawson and Russell, 2014)

Skhalta Dam located is in south-west Georgia. Due to landslide and erosion there is the deposition of alluvial sand, gravels and different sediments. It comprises of alluvium and lacustrine deposits as predominant material. It has clear variation in thickness across the valley and presence of a low strength layer. At centre of valley there is 50m thick alluvial deposits. In this scenario low strength foundation is necessary which can tolerate differential settlement on it. The minimum footprint of dam is required so that the treatment volume is decreased. Treatment volume is directly related to cost. Dam should be cost effective. So, faced symmetrical hardfill dam was selected to meet all those criteria.

The major problem in Skhalta dam is higher excavation depth. The depth of excavation is necessary to decrease for cost effectiveness of dam. There is the presence of lacustrine deposits and alluvial deposits with varying thickness. The lacustrine deposits of foundation material possess major threat to foundation stability. This is due to presence of low strength layer in alluvial deposits. The coefficient of permeability at each elementary portion of dam is varied. There are chances of differential settlement on deposits.

The presence of combination of alluvial and lacustrine deposits impose serious challenge towards the stability of dam. The lacustrine deposits have very low strength. It should be excavated and replace by suitable fill materials. To minimize the cost of replacement and treatment of soil deposits, dam having minimum foot print area is necessary. For this a face symmetrical hardfill dam is select as a best option. The correct assessment of material modulus of elasticity is necessary to predict the accurate settlement. With the help of 3D modelling of dam on three different conditions; end construction, reservoir full, and reservoir full and silted, we can predict the settlement on different occasions. Based on the settlement of model we can judge the degree of effectiveness of ground improvement techniques on dam. It helps to determine the need of further grout material in soil.

Case study IV: Evaluation and treatment of seepage problems at Chapar-Abad Dam, Iran (Uromeihy and Barzegari, 2007)

Chapar-Abad dam consists of alluvial soil of 60 metre thickness as foundation material. It has various degree of permeability on its layer. Rock on right abutment is laminated limestones which are weathered and fractured at surface. The rock at left abutment consists of metamorphosed limestone and shale. The potential of water leakage is more in right abutment than of left abutment. A remedial measure is proposed before the construction of dam. It is more than the dam height.

Dam is highly susceptible to seepage after water impoundment due to the presence of thick alluvium deposits. The permeability varies on its layer. The problem is to design an appropriate method to control the seepage which can match the complex features of the ground.

By using finite element software, we can estimate the seepage of dam. There are various ground improvement techniques available to calculate the seepage of dam. The choice between them depends upon the technical availability and operation cost. The grouting operation is to be perform in two rows. The upper rows are for foundation consolidation whereas lower rows are to form the grout curtain. Grout curtain produces a fruitful result when it is installed between alluvial deposits and impermeable core.

Case study V: Assessment and Presentation of a Treatment Method to Seepage Problems of the Alluvial Foundation of Ghordanloo Dam, NE Iran (Hedayati Talouki et al., 2015)

The maximum thickness of alluvial deposits in Ghordanloo dam is 60.50 meters. The dam foundation consists of two groups: lean clay with sand and low plasticity; and silty clay with low plasticity; and silty clayey gravel with sand by unified soil classification system. From the stratigraphic view point the site geology consists of Tirgan and Sarcheshme formation and Quaternary deposits. The Tirgan formation is ootilitic limestone with thin layers of marly limestone and marl. The Sarcheslme formation consist of highly weathered and fragmented shale with lime layers. And quaternary deposits consist of river bed deposits, alluvial terraces and the talus.

Ghordanloo dam is an earth fill dam having clay core height 46m, crest length 236m, and a reservoir capacity of 220 million cubic meters. Earthen dam is a flexible dam so no need to consider bearing capacity and deformation during its design. Seepage and the hydraulic gradient are the two factors which should be consider for proper water proofing of dam

The major consideration in the design is 60m thick alluvium deposits in dam axis. It has a various degree of permeability on its layer. The two major problems are seepage and hydraulic gradient. Attention is necessary on seepage and hydraulic gradient for water proofing of dam.

The combination of clay blanket and cutoff wall is an effective means to control seepage in weak foundations. Seepage and the hydraulic gradient are the two factors which should be consider for proper water proofing of dam. RCC dam is a rigid dam so need to calculate bearing capacity and deformation during its design. These can be found out by using SEEP/W software. From the results of SEEP/W software optimum depth of cutoff wall is find out.

2.19. Numerical Modelling of Dam

The numerical model has a huge advantage over a physical model. The complex geometry with considerations of different parameters can be modelled. The critical parameters for modelling can be found out with ease in numerical modelling through trial. The numerical model of dam can solve complex physical model comparatively faster. The numerical modelling of dam and foundation material can be done by different numerical modelling technique. The value of deformation, pore pressure, vertical stress, strain and seepage can be found out. The different numerical modelling approach are:

2.19.1. Finite Element Technique

Finite element analysis is a numerical method used to solve the engineering problems using array of mathematical techniques. The name comes from the fact that the methods subdivided the larger parts problem into smaller simpler parts called finite elements. In each element, there is a simplified relationship between loads and displacements. The equations that modelled these finite elements are solved and reassemble back into larger system of equation that modelled the entire problem. The continuity equations links all of the equations that are in all of the elements. When we linked up together, we end up in single matrix equation. A boundary conditions is applied that specifies what points are known to displacements and loads. Finally, in post-processing it returns to each element to interpolate local displacements and stresses. Finite element analysis can solve boundary value problems like stress analysis, heat transfer, fluid flow and electric or magnetic potential.

2.19.2 Finite Difference (FD) Technique

The fundamental of the FDM is the discretization of the domain. The variables involved in the equation may be temporal or spatial variables. It means solution of the equation yields the values of the unknown variables at particular locations at particular time. FDM involves the breakdown of continuous equation into discrete value of the variables at different points called as nodes. The calculation proceeds in nodal form. The spatial domain is represented by grid lines and hence the intersection of grid lines are the nodal points.

Domain is the area or boundary within which the analysis of any physical process is done. We represent the physical processes by the physical equations and try to solve them in order to determine the actual phenomenon happening inside the domain and interpret it to reach a conclusion. The governing equations of complex process are mostly in differential equation form. Simple differential equations may be solved analytically. However, not all complex equations may not be solved as such. Approximate method of solving the equation should be adopted. This is the situation where FDM comes into play.

For the commencement of the numeric calculations, the value of the variables known at the start of computation are assigned respectively. This is known as initial condition. At any other point within domain, the value of any parameter at any time may be known beforehand the calculation. This is known as boundary condition. Thus, initial and boundary conditions are assigned before the computation actually begins.

There are various approaches for the numerical calculation along the grid lines of the domain. Some of the fundamental approaches as described by (Stephenson and Meadows, 1986) are as shown in Figure 2.7:

a) Forward difference method

For spatial variable,

$$\frac{\partial U}{\partial y} = \frac{U^{Y+\Delta Y,t} - U^{Y,t}}{\Delta Y} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \textit{Equation } 2.3$$

For temporal variable,

$$\frac{\partial U}{\partial t} = \frac{U^{Y,t+\Delta t} - U^{Y,t}}{\Delta t} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \textit{Equation } 2.4$$

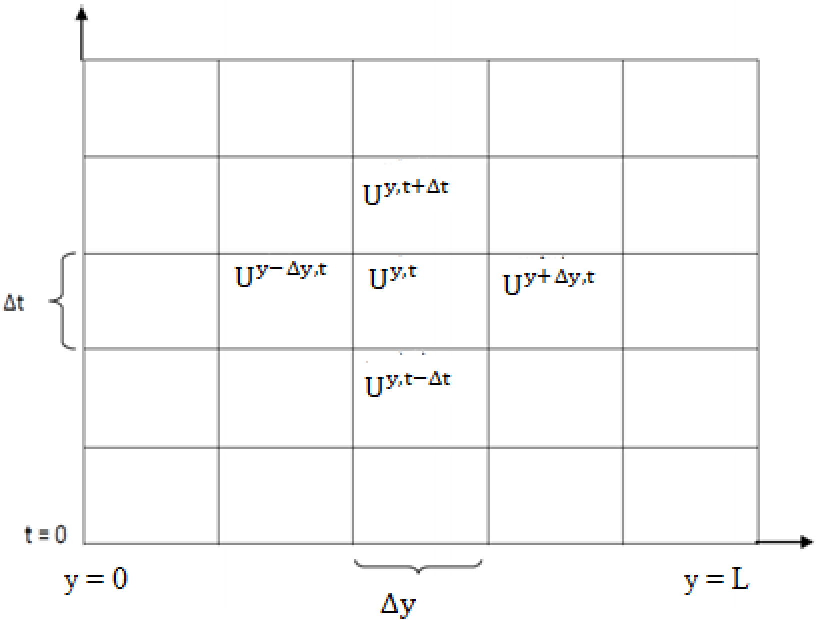

Figure 2. 8: Rectangular mesh showing nodal points used in the finite difference technique.

b) Backward difference method

For spatial variable,

$$\frac{\partial U}{\partial y} = \frac{U^{Y,t} - U^{Y-\Delta Y,t}}{\Delta Y} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \textit{Equation } 2.5$$

For temporal variable,

$$\frac{\partial U}{\partial t} = \frac{U^{Y,t} - U^{Y,t-\Delta t}}{\Delta t} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \textit{Equation } 2.6$$

c) Central difference method

For spatial variable,

$$\frac{\partial U}{\partial y} = \frac{U^{Y+\Delta Y,t} - U^{Y-\Delta Y,t}}{2\Delta Y} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \textit{Equation } 2.7$$

For temporal variable,

$$\frac{\partial U}{\partial t} = \frac{U^{Y,t+\Delta t} - U^{Y,t-\Delta t}}{2\Delta t} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \; Equation\ 2.8$$

3. METHODOLOGY

3.1. Introduction

This chapter explain the method to study and simulate the performance of alluvial dam foundation. The performance of dam foundation is analysed at natural and improved alluvial soil conditions. The borehole data, MASW survey report, field test and laboratory test result are collected which contains alluvial deposits as foundation material. The value of mechanical parameters at natural conditions is calculated from MASW survey report, laboratory and field test results. And mechanical parameter is calculated for improved soil conditions from borehole data, field test, laboratory test and MASW survey reports. To validate the method presented two-dimensional finite difference software and finite element software is used. It provides the value of deformation, vertical stress, porewater pressure and seepage at natural and improved soil conditions.

The methodology followed are:

1. Desk Study and Literature Review
2. Derivation of Analysis Procedure and Assumptions
3. Determine soil parameters in natural soil conditions for analysis. (K, ρ, G, k, e, VWC)
4. Derivation of material property for improved soil conditions. (K, ρ, G, k, e, VWC)
5. Modelling homogenous and heterogenous material distribution at natural soil conditions.
6. Modelling of Dam foundation for Natural soil conditions.
7. Determine vertical deformation, vertical stress, pore-pressure, and seepage at natural soil conditions.
8. Modelling homogenous and heterogenous Material Distribution at improved soil conditions.
9. Modelling of Dam foundation for improved soil conditions.
10. Determine vertical deformation, vertical stress, pore-pressure, seepage at improved soil conditions.
11. Modelling of seepage through gravity dam.
12. Presentation of results and discussion on methodology and results. (Chapter four)

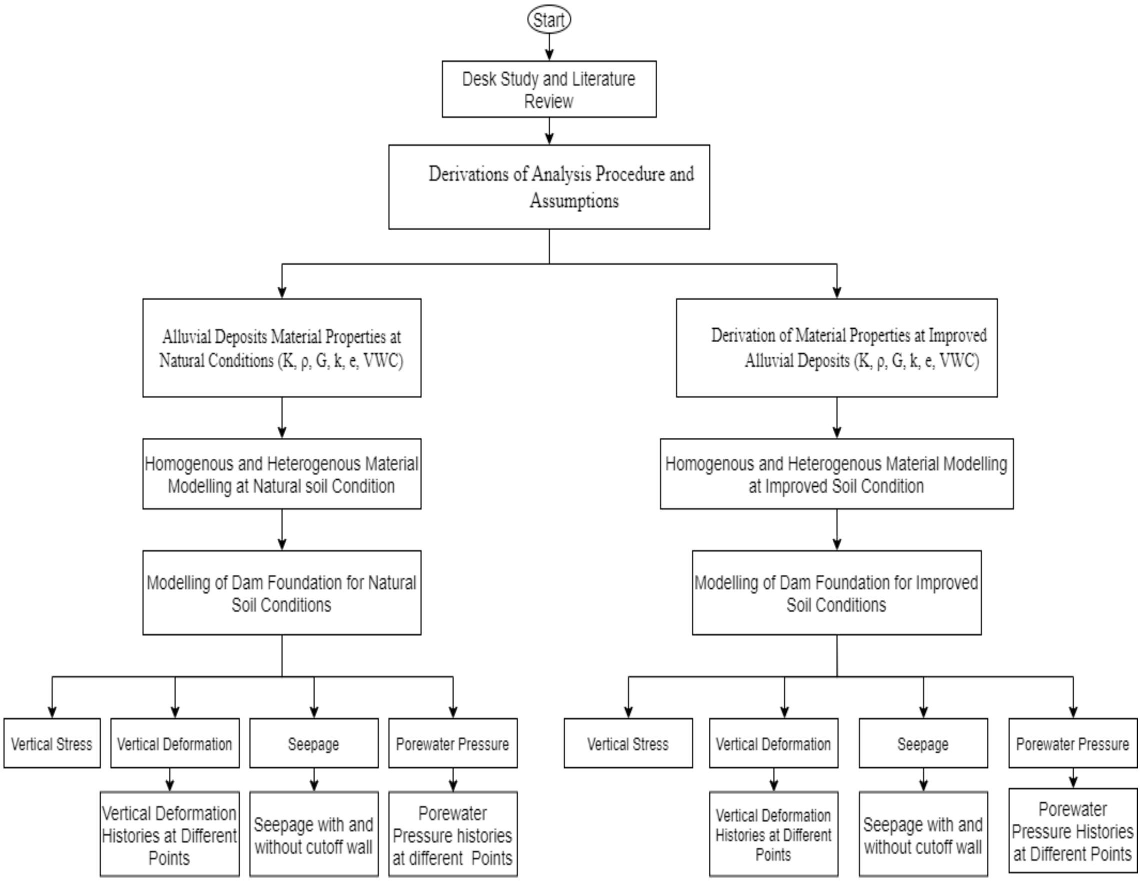

Figure 3. 1: Flow chart of Methodology

3.2. Desk Study and Literature Review

The necessary books, literature, research paper is collected and studied. The FDM and FEM software is collected. The report from sites containing alluvial deposits as foundation material is collected. It contains sieve analysis, permeability test, MASW survey reports and borehole logs.

3.3. Derivation of Analysis Procedure and Assumptions

The main aim of this section is to finalise the exact procedure to be followed for the analysis of dam foundation. The analysis needs to be done at natural and improved alluvial soil deposits. For this it is necessary to determine the mechanical parameters of soil at natural and improved soil conditions. The MASW survey report, field test results, laboratory tests results and borehole log having alluvial deposits as a subbase material is collected from two different locations. Borehole log shows the predominant material consists of Boulder, Gravel, Silt and Sand To analyse these results one important assumptions is made. All the foundation material is categorized into two portion fine fraction and coarse fraction.

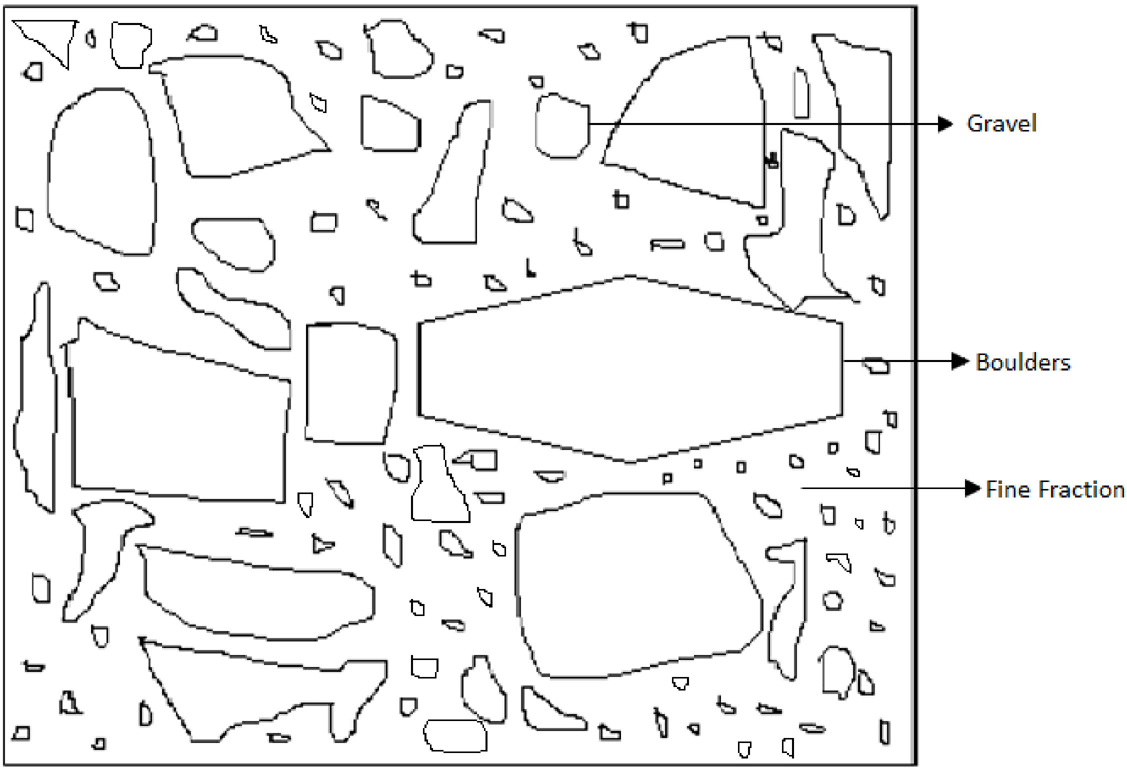

Figure 3. 2: Alluvial Deposits Containing Heterogenous mixture of Gravel, Boulder and Fine Fraction

Figure 3.2 simulates the actual subsurface condition of field. Fine fraction is the quantity of fine particles except boulder and gravel in an alluvial deposit. It is of sand and silt. Coarse fraction is the quantity of boulders and gravels in an alluvial deposit. MASW reports (Table 3.1) gives the values of mechanical parameters (E, G, and ρ) of soils. By analysing the results of MASW survey report and borehole log percentage fine fraction relationship between mechanical parameters of soil is formulated. These equations can provide the value of mechanical parameters at natural and improved soil conditions. This relation is used to predict the mechanical parameters after the improvement of soil with consolidation and compaction grouting which decreases the percentage of void with fines.

The effect of the mechanical parameters for the stability of gravity dam is to be studied. To study this, soil material is modelled as homogeneous and heterogenous material consisting different fraction of boulders and fines for natural and improved soil conditions. The validation of presented method can be done through use of FDM and FEM software at natural and improved soil conditions. FDM and FEM will give the value of deformation, vertical stress, porewater pressure and seepage at natural and improved soil conditions.

3.4. Determine soil parameters in natural soil conditions for analysis. (K, ρ, G, k, e, VWC)

The soil investigations reports of Rolwaling Khola Hydropower Project and Kimanthanka hydroelectric project are collected from NEA engineering company limited. Both sites have alluvial deposits as a foundation material and there was negligible amount of clay presence in alluvial deposits. The subsurface geological survey of Kimanthanka Arun Hydroelectric Project was conducted by multi-channel analysis of the surface waves (MASW) at major structural sites of the project such as dam axis, intake, powerhouse and tailrace area are collected. A number of bore hole done at major structural sites was collected from both sites. The bore hole of Kimanthanka was drilled vertically up to the depth of 60.00m whereas of Rolwling khola was drilled vertically to a depth of 30m. Field test results and laboratory tests results of both sites are collected. The value of (K, ρ, G, k, e, and VWC) is found out at natural state.

The different modelling case of gravity dam done to find out deformation, deformation histories at various depth, vertical stress, porewater pressure, porewater pressure histories and seepage are as follows:

i. Linear Distribution of Homogenous Materials

(a) Linear distribution of homogenous material at 80, 60 and 40 percentage fine fraction

ii. Heterogenous random distribution of Material

(a) Random Distribution of Heterogenous Material at 80 percentage Fine Fraction

iii. The different modelling case of gravity dam done to calculate seepage are:

(a) Seepage at natural soil conditions.

(b) Seepage at natural soil conditions with provision of 10m cutoff wall.

3.5. Derivation of material property for improved soil conditions. (K, ρ, G, k, e, VWC)

The borehole data shows the lithological characteristics of subsurface which is predominantly consists of sands, gravel, boulder of varying size. And also, the sub-surface geological condition of the project area done by multi-channel analysis of the surface waves (MASW) at major structural sites of the project is collected. MASW calculates the shear wave velocity profile of sub-surface ground condition.

Table 3. 1: MASW profile showing subsurface condition (25m depth)

Depth(m)		**Thickness (m)**	**V_s (m/s)**	***Density (Kg/m^3)**	**Young's Modulus (N/m^2)**	**Shear Modulus (N/m^2)**	**Bulk Modulus (N/m^2)**
0	3.97	3.97	226	1430.62	196908054	72747179	223818821.5
3.97	4.77	0.8	356	1531.34	525607915	194184506	597440996.8
4.77	5.73	0.96	331	1518.51	454004838	166671652	548210841.6
5.73	6.88	1.15	419	1585.76	757980296	278265379	915261208
6.88	8.26	1.38	398	1570.12	678501576	249087607	819290652.4
8.26	9.92	1.66	371	1549.22	580844189	213236185	701369353.1
9.92	11.91	1.99	420	1637.87	812115500	289058059	1421202125
11.91	14.3	2.39	645	1853.45	2184532012	770841895	4385448014
14.3	17.17	2.87	394	1662.66	737699970	257973145	1751422678
17.17	25	7.83	652	1898.89	2306216006	806482067.3	5475341168

Based on shear-wave velocity profile, the dynamic parameters of soil: density, moduli of elasticity of each layer has been correlated. From the 60m depth of borehole log, the distribution of percentage fine fraction and percentage coarse fraction per meter of layer is found out. Then, the fine fraction percentage of borehole log is compared to shear wave velocity profile of MASW survey. The relation between percentage fine fraction and dynamic properties of alluvial deposits is calculated from below table.

Table 3. 2: Percentage Fine Fraction Relationship with Moduli of Elasticity and Density

S. N	**Percentage fine contents (%)**	**Shear Modulus of Elasticity (1X10^7 N/m2)**	**Bulk Modulus of Elasticity (1X10^7 N/m2)**	**Density (Kg/m3)**
1	20	38.70	193.91	1675.6
2	25	35.02	173.5575	1679.3
3	30	31.5	154.165	1671.6
4	35	28.17	135.73	1654.2
5	40	25	118.26	1628.9
6	45	21.9	101.75	1597.6
7	50	19.13	86.2	1562
8	55	16.45	71.6	1524
9	60	13.9	57.9	1485.3
10	65	11.58	45.30	1447.7
11	70	9.4	33.58	1413.1
12	75	7.37	22.83	1383.2
13	80	5.51	13.04	1359.9

An alluvial deposit consists of fine fraction and coarse fraction. The proportion of fine fraction and coarse fraction governs the mechanical parameters K, G and ρ of soils which affect the strength and bearing capacity of deposits. By plotting the value of percentage fine fraction on x-axis and moduli of elasticity and density on y-axis, a relation is established from curvilinear graph. The established relation of percentage fine content with bulk modulus of elasticity, shear modulus of elasticity and density is shown in equation below (3.1), (3.2) and (3.3) and below figure 3.2, 3.3, and 3.4.

$$y = 0.0033x^2 - 0.8831x + 55.039 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad Equation\ 3.1$$

where, y = Shear modulus of elasticity, x = percentage fine fraction

$$y = 0.0024x^3 - 0.4092x^2 + 15.498x + 1510.1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad Equation\ 3.2$$

where, y = density, x = percentage fine fraction

$$y = 0.0192x^2 - 4.9345x + 284.92 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad Equation\ 3.3$$

where, y = Bulk modulus of elasticity, x = percentage fine fraction

These above three equations can be used to find out the mechanical parameters of alluvial deposits with respect to its percentage fine fraction.

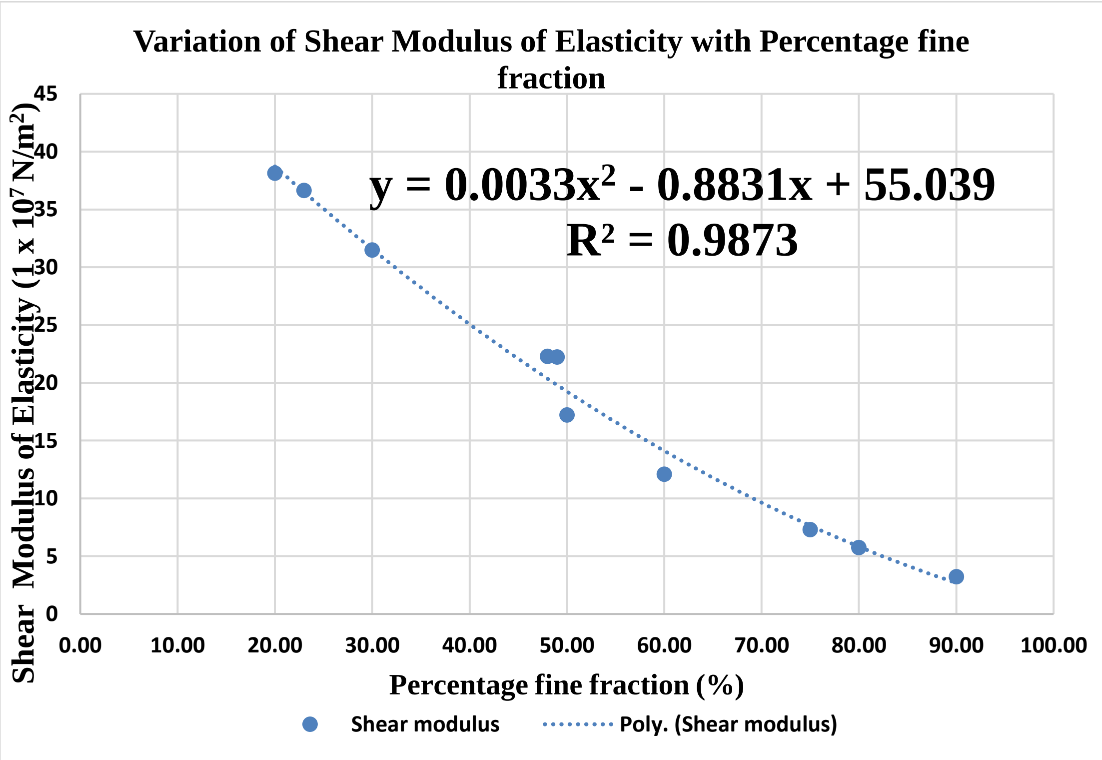

Figure 3. 3: Variation of Shear Modulus of Elasticity with Percentage Fine Fraction in an Alluvial Deposits

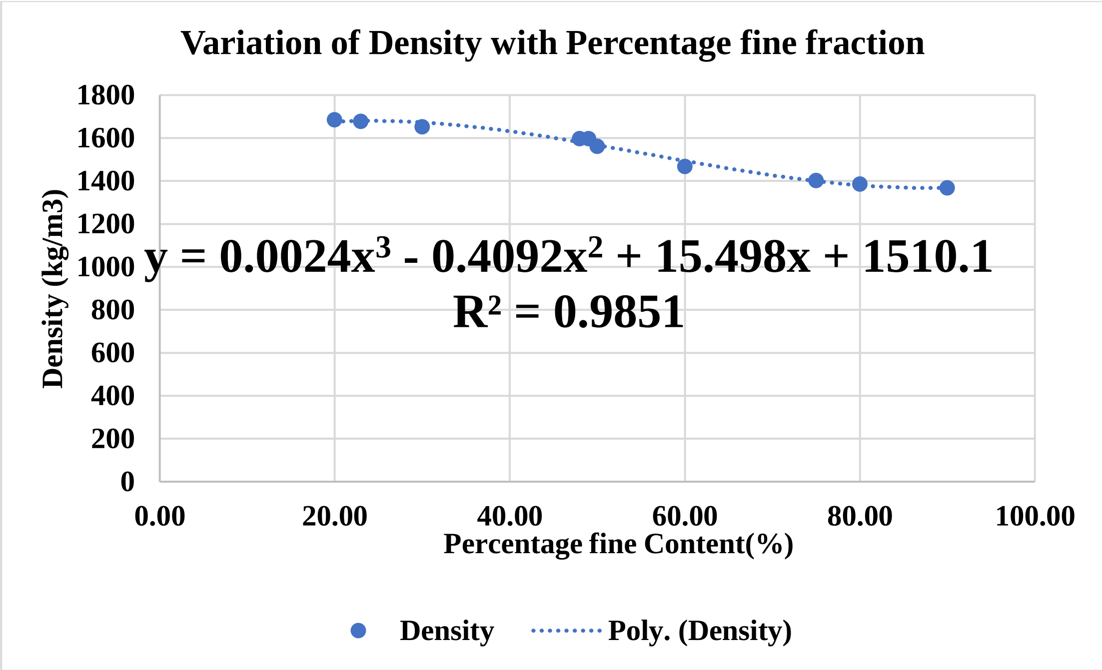

Figure 3. 4: Variation of Density with Percentage Fine Fraction in an Alluvial Deposits

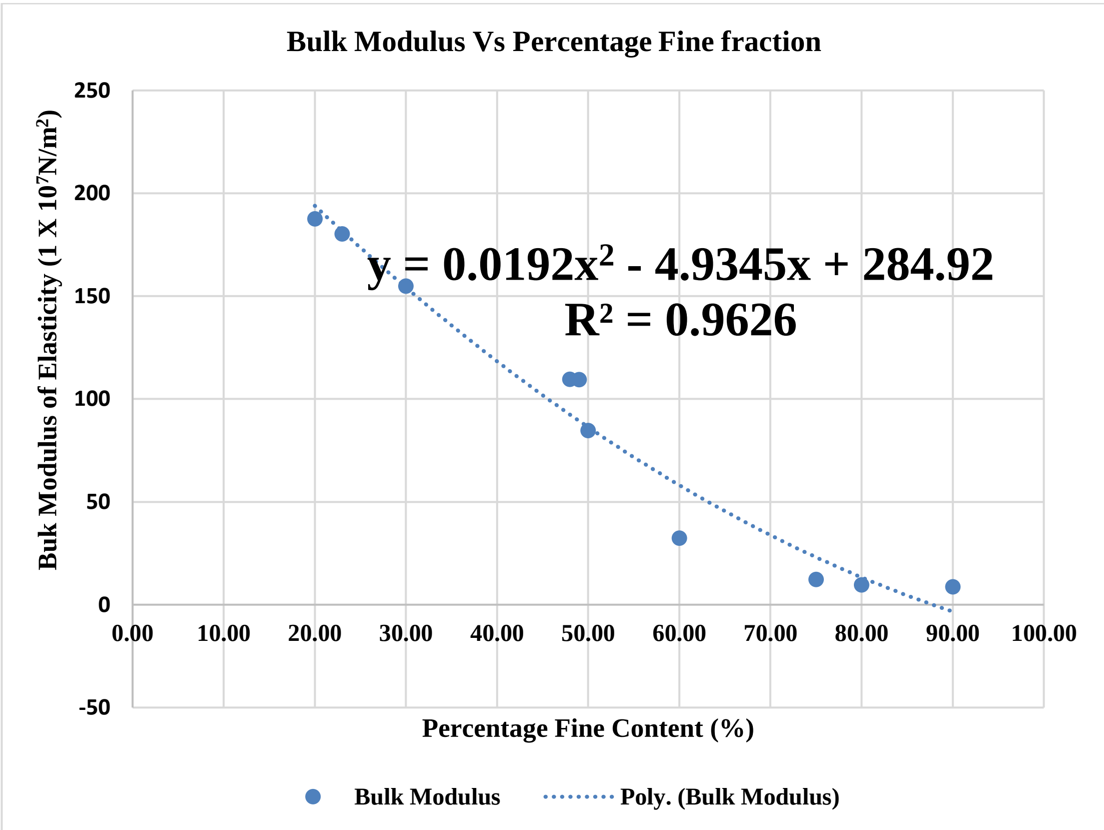

Figure 3. 5: Variation of Bulk Modulus of Elasticity with Percentage Fine Fraction in an Alluvial Deposits

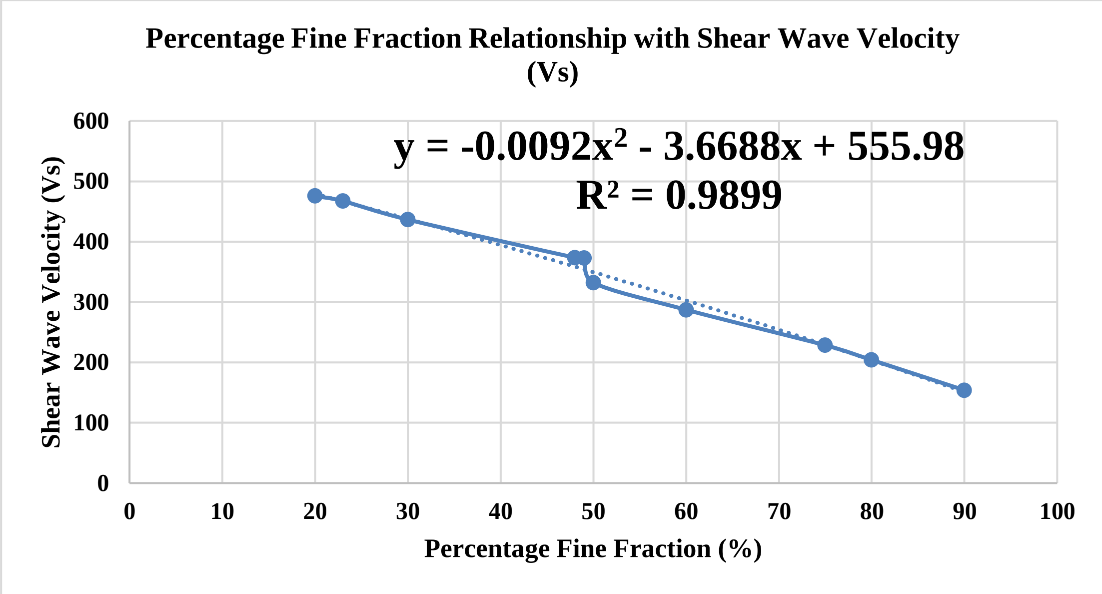

Figure 3. 6: Variation of Shear Wave Velocity with Percentage Fine Fraction in an Alluvial Deposits.

3.5.1 Estimation of permeability

The permeability is calculated as follows:

$$F = \frac{2\pi L}{\log_e[\left(\frac{2L}{D}\right) + \sqrt{1 + \{\frac{2L^2}{D}\}}]} \ldots\ldots\ldots\ldots\ldots\ldots \; Equation\ 3.4; \text{(British Standards Institution, 2015)}$$

Where,

$$K = \frac{q}{F} * H_c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; Equation\ 3.5$$

K = coefficient of permeability

L = Test Length, D = Diameter of the hole

Q = Rate of flow, litre/sec, H_c = Constant head, F = Intake factor

Table 3. 3: Field Permeability Test Results

S.N.	Drill Hole No.	Depth(m)	Test time (Sec)	Discharge, CC	Flow rate, (litre/min)	Permeability (cm/sec)
1	DH-1	9.8-10	600	300000	8.33	0.556
2	DH-1	9.8-10	600	304000	8.44	0.563
3	DH-1	9.8-10	600	303000	8.42	0.561
4	DH-1	9.8-10	600	340000	9.44	1.96
5	DH-1	20.9-21	600	342000	9.5	1.97
6	DH-1	20.9-21	600	340000	9.44	1.96
					Average of DH-1	= 1.26
1	DH-2	10.4-10.5	600	403000	11.19	2.323
2	DH-2	10.4-10.5	600	410000	11.39	2.364
3	DH-2	10.4-10.5	600	410000	11.39	2.364
4	DH-2	19.9-20	600	410000	11.39	2.364
5	DH-2	19.9-20	600	185000	5.14	1.067
6	DH-2	19.9-20	600	182000	5.06	1.049
					Average of DH-2	= 1.92
1	DH-3	10.25-10.5	600	480000	13.33	0.612
2	DH-3	20.8-21	600	1480000	13.70	0.914
					Average of DH-3	= 0.763

As we can see from above table there seems a variation in permeability in between 0.6 to 1.97 cm/sec. Therefore, taking the average of three bore-hole section.

Permeability = $\frac{(DH-1)+(DH-2)+(DH-3)}{3} = 1.314 cm/sec$

3.5.2. Permeability as a function of grain size distribution

Kozeny–Carman equation calculate value of permeability as a function of grain size distribution. The Kozeny–Carman equation is, actually, a special form of Darcy's law, so it is applicable for every possible natural sample of porous media. For granular material composition consists of natural uniform sand and gravel modified Kozeny–equation equation gives the better results.

$$K = \frac{\rho g}{\mu} \frac{n_e^3}{180(1-n_e)^2} {D_m}^2 \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \mathit{Equation}\ 3.6$$

$$K = 0.625\ \frac{n_e^3}{180(1-n_e)^2} {D_m}^2 \left(\frac{m}{s}\right) \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \mathit{Equation}\ 3.7$$ (Urumović and Urumović, 2014)

Where, ρ represents the density (ML^{-3}) represents the viscosity of water ($ML^{-1}T^{-1}$), g is acceleration due to gravity (MLT^{-2}), n_e is porosity of deposits, D_m is the diameter of mean grain.

Where, $\frac{\rho g}{\mu} = 0.625$; (Urumović and Urumović, 2014)

Using above formula,

For k = 1.314 cm/sec, n = 0.69

The relationship between void ratio and porosity is:

$$e = \frac{n}{1-n} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \mathit{Equation}\ 3.8$$

from above equation, e = 2.23

The value of permeability is estimated for decrease in percentage porosity and void ratio from natural conditions as a function of grain size distribution. Grouting decreases the fine fraction percentage in a deposit. Consolidation grouting and compaction grouting decreases the fine fraction by filling up voids through penetrating the alluvial deposits spaces and displacing the nearby soil particles. It increases the overall stiffness of deposits. The stiffer the deposits, lesser would be the presence of voids and more impermeable the material. Less permeable the material, lesser would be the value of seepage. Using the above equation of permeability (Urumović and Urumović, 2014), sieve analysis report of deposits, the value of permeability at decreasing fine fraction is found out. After 20, 40, 60, 60 and 80 percentage decrease in porosity the value of permeability is found out to be 0.000294m/sec, 0.000725m/s, 0.000140m/s and 0.0000124m/sec from above equation. The value of volume compressibility for sand is determined from literature as 4 X 10^{-5} /KPa. Volumetric water content is a numerical measure of soil moisture which is the ratio of water volume to soil volume. The volumetric water content is one way to describe a soil's moisture content. It is calculated as the ratio of the volume of water for a given volume of soil. The volumetric water content (θ) is the product of the soil saturation and the porosity.

Table 3. 4: Value of Permeability at different Porosity

S.N.	Void ratio(e)	Porosity(n) = volumetric water content function (for fully saturated soil)	Permeability (k) (m/sec)
1.	2.22	0.69	0.0134
2.	1.23	0.552	0.000294
3.	0.70	0.414	0.000725
4.	0.38	0.276	0.000140
5.	0.16	0.138	0.0000124

$$\mathrm{n} = \frac{\mathrm{V_V}}{\mathrm{V_T}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; Equation\ 3.9$$

$$\mathrm{S} = \frac{\mathrm{V_W}}{\mathrm{V_V}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; Equation\ 3.10$$

$$\theta = \mathrm{S\ x\ n} = \frac{\mathrm{V_W}}{\mathrm{V_T}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; Equation\ 3.11$$

Where, n = porosity, S = soil saturation, θ = volumetric water content, V_V= volume of voids, V_T= Total volume, V_W= volume of water

3.6. Performing homogenous and heterogenous material distribution at natural soil conditions.

a. Material modelling of homogenous materials

The material modelling considering linear distribution can be done from three equation shown in figure 3.3, 3.4 and 3.5. These equations show the percentage fine fraction relationship with moduli of elasticity and density.

b. Material modelling of heterogenous materials

From borehole data, we can find out random heterogenous distribution of foundation material. Heterogenous distribution is an uneven distribution of fine fraction and coarse fraction in a deposit of soil. There was non-linear variation of material properties on its cross-section. To represent the field condition more realistically, soil material is modelled as heterogenous material consisting different fraction of boulders and fines. The material modelling of random heterogenous material is performed using finite difference method at plane-strain condition. The behaviour of material is considered as linear elastic-perfectly plastic. To represent this material model Mohr-coulomb material model is considered for analysis. The width and depth of alluvial deposits are taken to be 60 and 30m respectively. The three sides of boundary are fixed and surface is free. During heterogenous material modelling the maximum and minimum value of E, G, and ρ is taken from above derived percentage fine fraction relationship. Percentage fine fraction and coarse fraction are randomly distributed in depth and width. The mechanical parameters of percentage fine fraction and coarse fraction are simulated in terms of bulk modulus of elasticity (E), shear modulus of elasticity (G) and density (ρ). The figure 3.7, 3.8, and 3.9. presents the simulation of the 80-percentage fine fraction and 20-percentage boulders in an alluvial deposit. The FDM method is used to simulate the value of mechanical parameters in terms of K, G, and ρ. Different values of K, G and ρ at 80-percentage fine fraction are randomly distributed to the model zones.

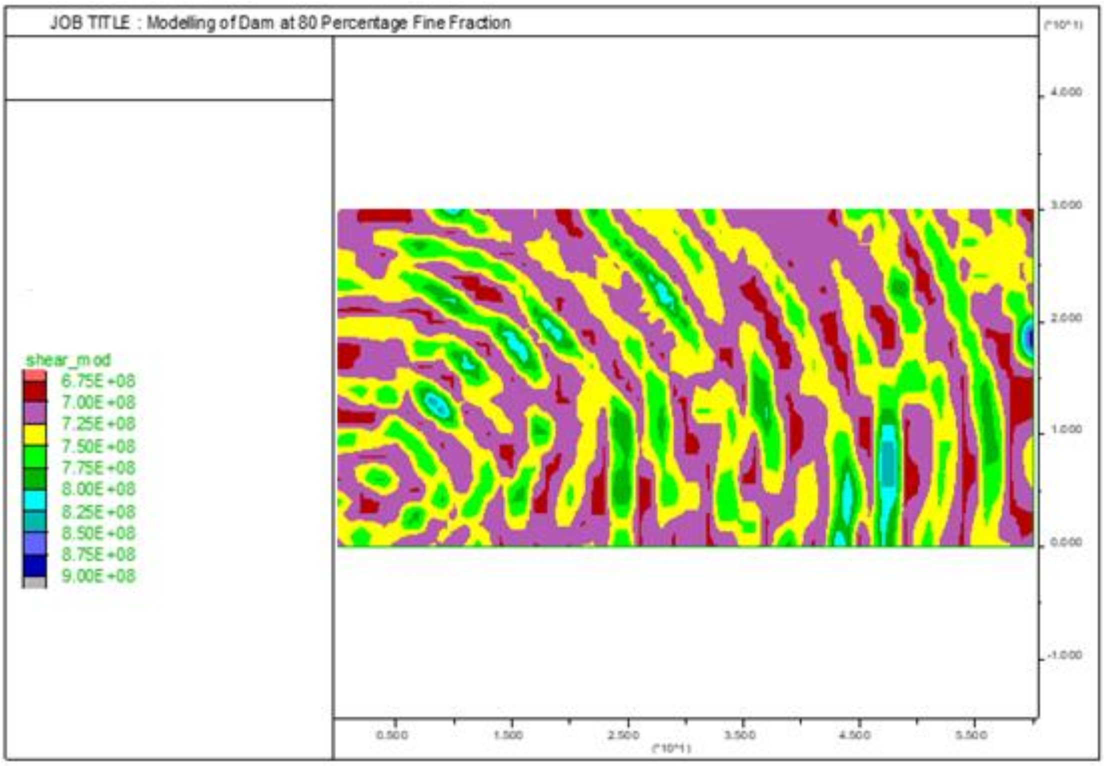

Figure 3. 7: Distribution of Shear Modulus of Foundation Material at 80 percentage Fine Fraction

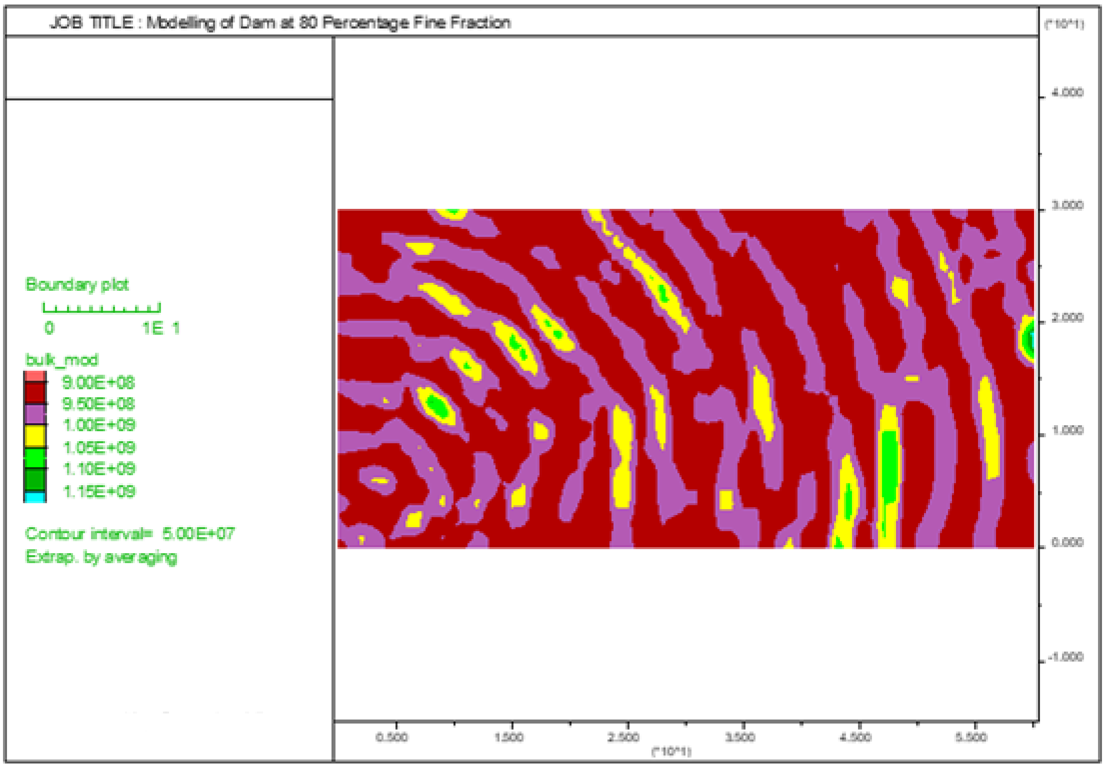

Figure 3. 8: Distribution of Bulk Modulus of Foundation Material at 80 percentage Fine Fraction

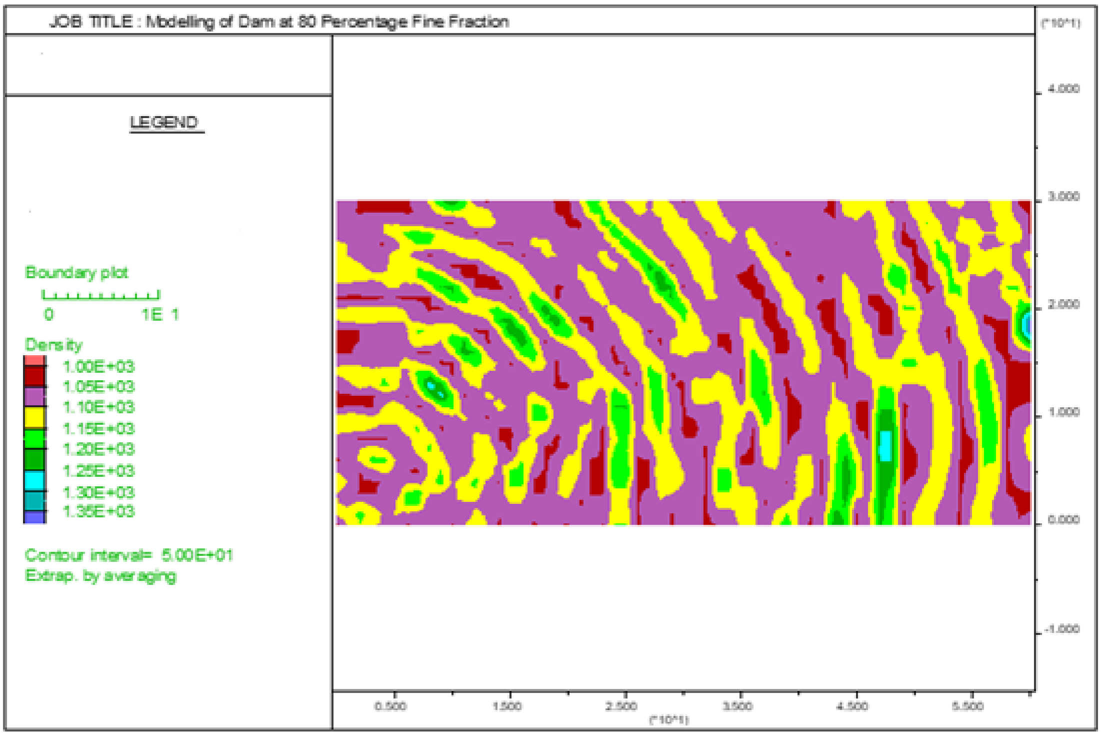

Figure 3. 9: Distribution of Density of Foundation Material at 80 percentage Fine Fraction

3.7. Modelling of Dam foundation for Natural soil conditions.

The total height and base width of gravity dam are 27m and 20m as shown in below figure:3.10. The sum of vertical and horizontal forces is found to be 4218.05KN and 1883.52 KN. The resisting and overturning moment are calculated 75562.94KNm and 35333.78KNm. Total stress on toe and heel are 240640 N/m^2 and 181166 N/m^2. The eccentricity of dam is found to be 0.47. The factor of safety against overturning and sliding are estimated more than 2 and 1.5. Thus, the stability of dam is assured from perspective of sliding and overturning. Now, it needs to be assured on bearing capacity for its stability.

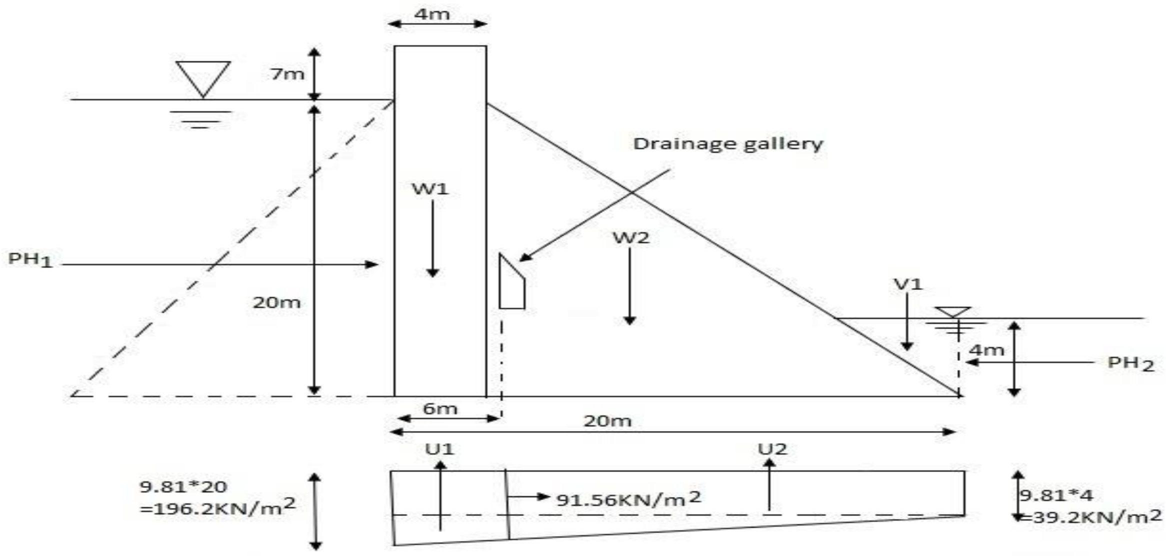

Figure 3. 10: Gravity Dam

For modelling, the thickness and width of subsurface material is taken as 30m and 60m at plane-strain condition. To transfer the dam load to foundation a rigid plate having high stiffness of 1m thickness is put at the surface of material. The bottom and sides of model is fixed so that it does not deform during modelling. The alluvial deposits are modelled as homogeneous and heterogenous material consisting different fraction of boulders and fines. The material modelling is done through Mohr-Coulomb material model where Soil-Gravel: mixture of gravel and sand is chosen. Then, the deformation, vertical stress distribution and porewater pressure at linear distribution and heterogenous foundation material will be found out. Also, the deformation histories and porewater pressure histories at linear depth with different timesteps will be found out for both linear and heterogenous material.

Further, the deformation, vertical stress, porewater pressure, deformation histories at different time steps, and porewater pressure histories at different timesteps for improved soil will be find out for both linear and heterogenous distribution of material.

3.8. Determine vertical deformation, vertical stress, pore-pressure, and seepage at natural soil conditions.

After the development of all the parameters required for numerical modelling through the method developed above. The value of vertical deformation, vertical stress, pore-pressure and seepage is calculated at natural conditions. These values will be compared to results of numerical modelling after soil improvement. This process helps to check the method presented.

3.9. Performing homogenous and heterogenous Material Distribution at improved soil conditions.

3.9.1. Material Modelling at 35 Percentage Fine fractions

Below figure 3.11, 3.12, and 3.13 simulated the 35-percentage fine fraction and 65-percentage boulders in an alluvial deposit. The FDM method is used to simulate the value of mechanical parameters in terms of E, G, and ρ as shown in below figure 3.6, 3.7 and 3.8. There are 12 different values of E, G and ρ at 35 percentage fine fraction which is shown in legend of below figure.

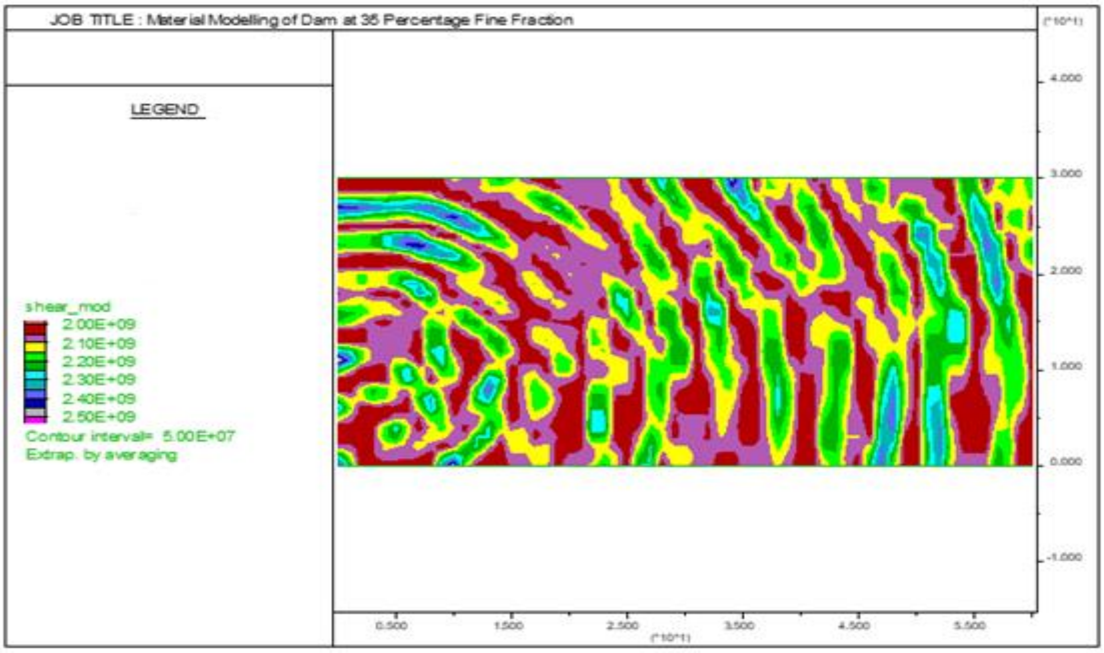

Figure 3. 11: Distribution of Shear Modulus of Foundation Material at 35 percentage Fine Fraction

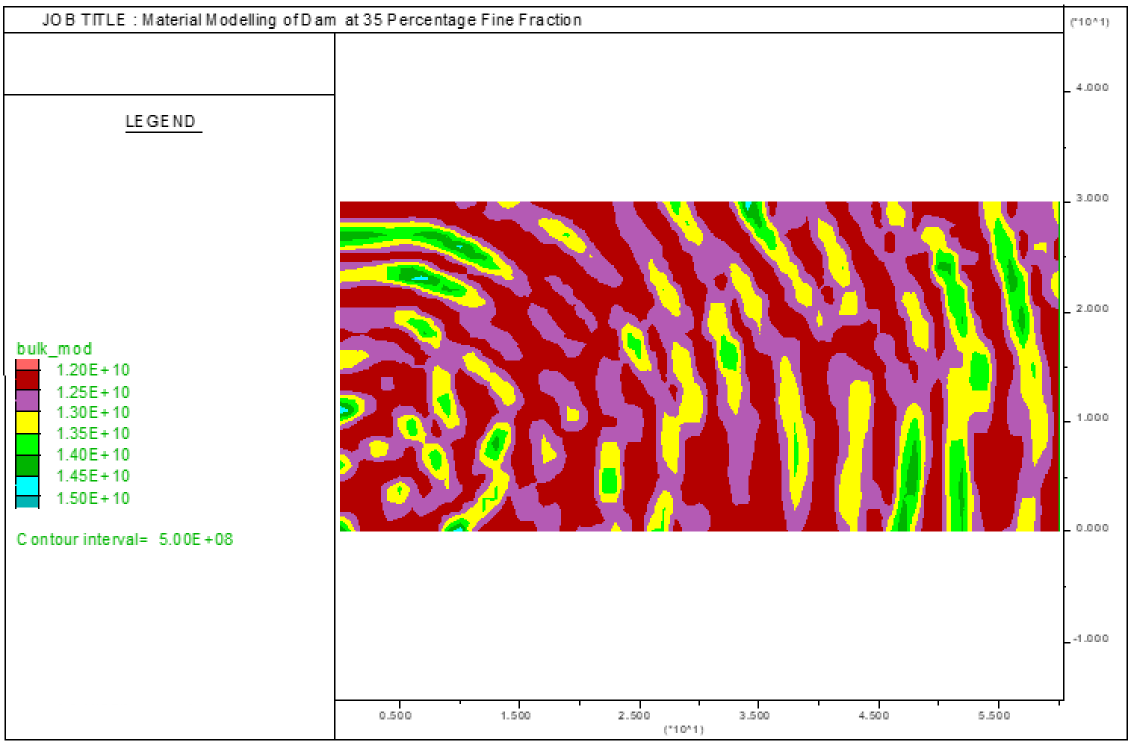

Figure 3. 12: Distribution of Bulk Modulus of Foundation Material at 35 percentage Fine Fraction

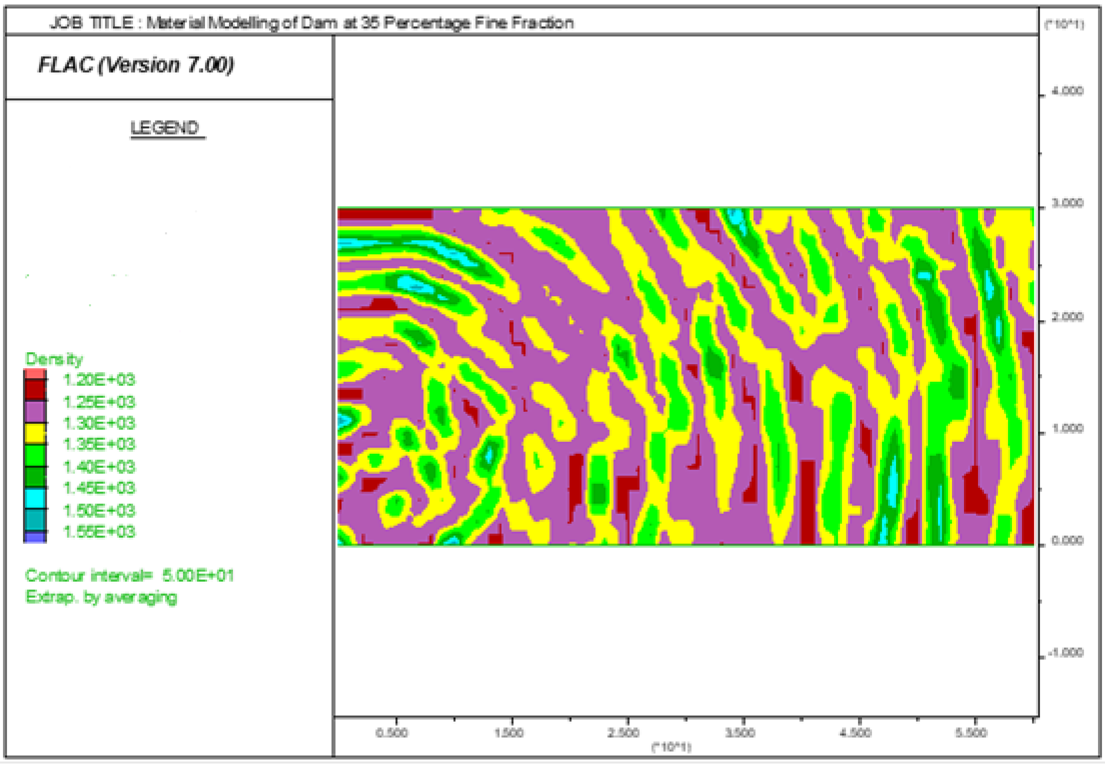

Figure 3. 13: Distribution of Density of Foundation Material at 35 Percentage Fine Fraction

3.9.2. Material Modelling at 20 percentage fine fraction

Below figure 3.14, 3.15, and 3.16 simulated the 20-percentage fine fraction and 80-percentage boulders in an alluvial deposit. The FDM method is used to simulate the value of mechanical parameters in terms of E, G, and ρ as shown in below figure 3.12, 3.13 and 3.14. There are 8 different values of E, G and ρ at 20 percentage fine fraction which is shown in legend of below figure.

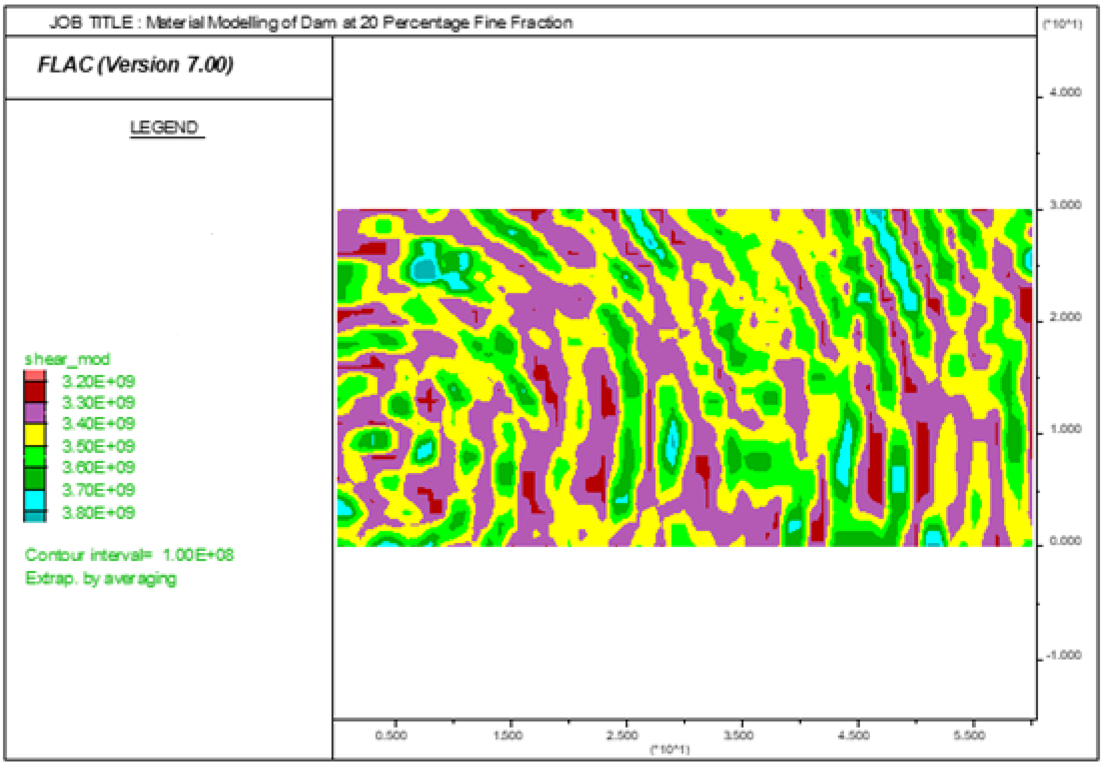

Figure 3. 14: Shear Modulus of Foundation Material at 20 Percentage Fine Fraction

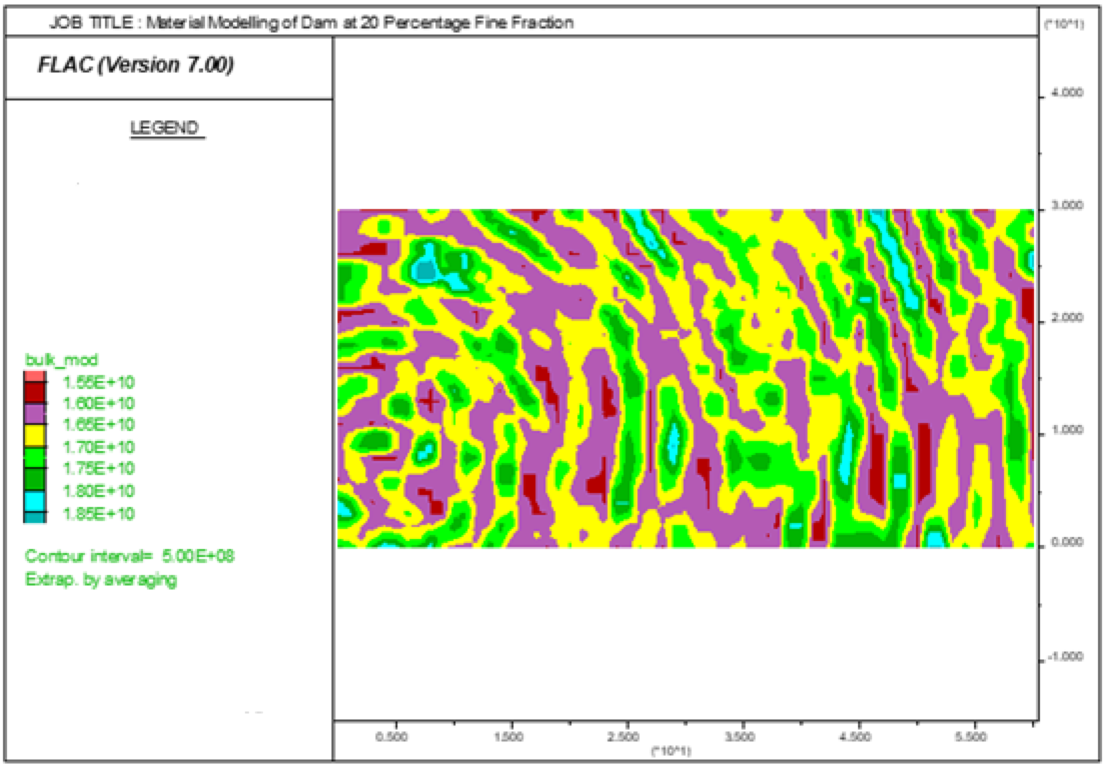

Figure 3. 15: Bulk Modulus of Foundation Material at 20 Percentage Fine Fraction

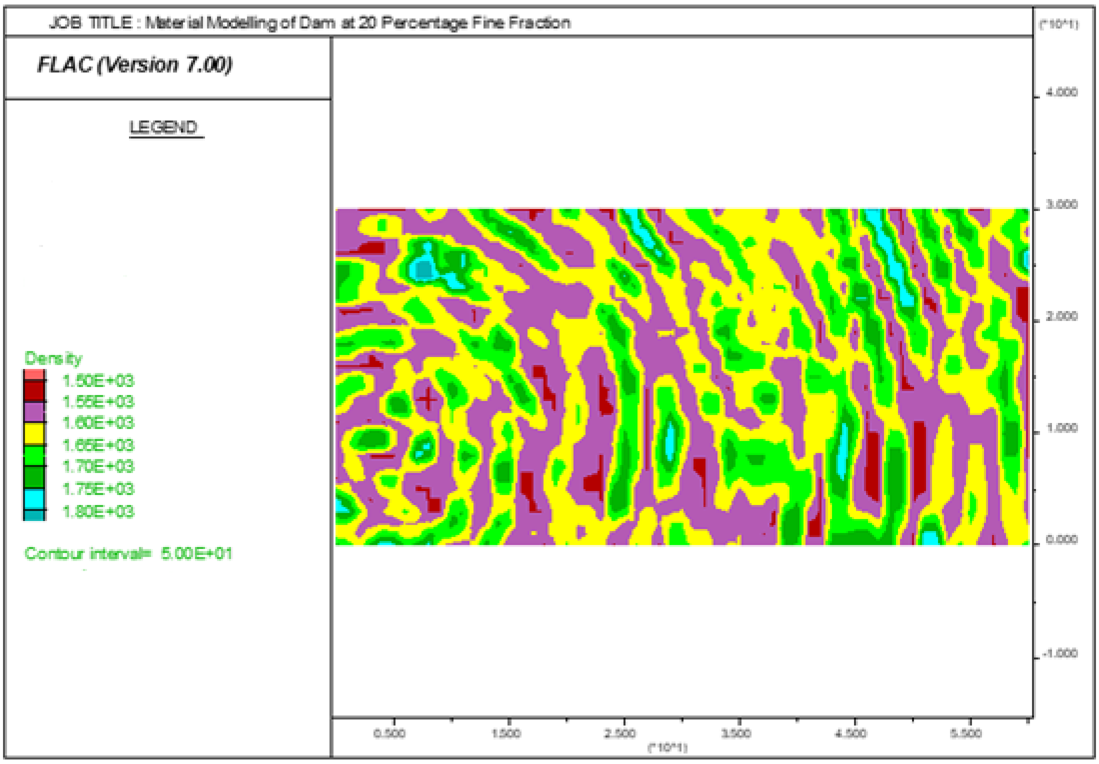

Figure 3. 16: Density of Foundation Material at 20 Percentage Fine Fraction

3.10. Modelling of Dam foundation for improved soil conditions.

The assumption is that, with the help of consolidation and compaction grouting the voids will be filled by cemented material such that the fine fraction will be decreased and consequently the stiffness of the material will be increased. This is represented by the increase in K, G and ρ values.

3.11. Determine vertical deformation, vertical stress, pore-pressure, seepage at improved soil conditions.

The different modelling case of gravity dam is necessary at improved soil conditions to find out value of deformation, deformation histories at various depth, vertical stress, porewater pressure, porewater pressure histories and seepage.

1. Linear Distribution of Homogenous Materials

   (a) Linear distribution of homogenous material at 80, 60 and 40 percentage fine fraction

   (b) Linear distribution of homogenous material at 50, 40 and 35 percentage fine fraction

   (c) Linear distribution of homogenous material at 30, 25 and 20 percentage fine fraction

   (d) Linear distribution of homogenous material at 20 percentage fine fraction

2. Heterogenous random distribution of Material

   (a) Random Distribution of Heterogeneous Material at 35 percentage Fine fraction

   (b) Random Distribution of Heterogeneous Material at 20 percentage Fine fraction

3. Different modelling case of gravity dam foundation to find out seepage

   (a) Seepage at improved soil conditions.

   (b) Seepage with provision of 10m cutoff wall at improved soil conditions.

3.12. Comparing the results of numerical modelling to verify the methodology presented.

The results of numerical modelling done by FDM and FEM software can be compared and analysed at natural and gradual improved condition. By comparing the different results at natural and improved conditions, we can verify the method presented.

4. RESULTS AND DISCUSSIONS

4.1 Introduction

The stability of dam is checked and material modelling of alluvial deposits is performed considering homogenous and heterogenous distribution. Deformation histories, vertical stress, excess porewater pressure on foundation material is determined considering linear variation and heterogenous random distribution. Seepage on foundation material is determined by modelling a 27m height gravity dam above it on natural and improved soil conditions.

4.2 Stability Analysis of Gravity Dam

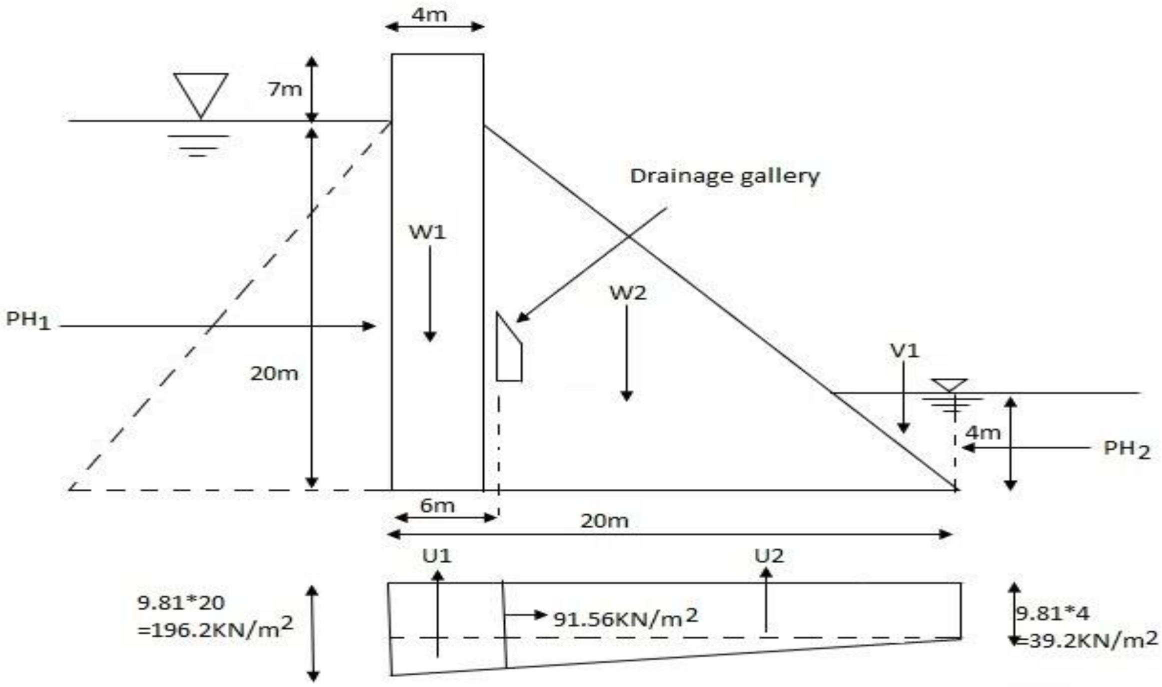

Figure 4. 1: Gravity Dam

Total height of dam = 27m

Freeboard = 7m

Minimum base width of dam = $\frac{H}{\sqrt{G-K}} = \frac{20}{\sqrt{2.5-0.7}} = 15m$ So, base width be more than 15m.

Where, Specific gravity (G) of soil = 2.5 and Seepage coefficient(K) = 0.7

Base width of dam = 20m

Total height of dam at do2wnstream = 2m

Drainage gallery from heel = 6m

Unit weight of Roller compacted concrete = $24\frac{KN}{m^3}$

Maximum vertical stress @ heel and toe of dam:

$\frac{\sigma_{heel}}{\sigma_{toe}} = \frac{\sum V}{B}\left(1 \pm \frac{6e}{B}\right)$

Table 4. 1: Stability analysis Calculations

SN.	Force (KN)	F_V (KN)	F_H(KN)	LA from Toe (m)	Resisting moment "M_R"(KN-m)	Overturning Moment "M_O"(KN-m)
1.	W_1	2592		17	44064	
2.	W_2	3360		9.33	31360	
3.	V_1	44.93		0.76	34.3	
4.	PH_2		78.48	1.33	104.64	
5.	U_1	-863.3		17.36		14986.54
6.	U_2	-915.6		8.77		7260.70
7.	PH_1		1962	6.67		13086.54
	Total	$\sum F_V$ =4218.05	$\sum F_H$ = 1883.52		$\sum M_R$ =75562.94	$\sum M_O$ =35333.78

Location of resultant force from toe ($\bar{x}$) $= \frac{\sum M_R - \sum M_O}{\sum F_V} = \frac{75562.94-35333.78}{4218.05} = 9.53\text{m}$

Eccentricity of dam: e = $\frac{20}{2} - \bar{x} = 10 - 9.53 = 0.47\text{m}$ is less than $\frac{B}{6} = \frac{20}{6} = 3.33\text{m}$. Ok...

We have,

$$\frac{\sigma_{heel}}{\sigma_{toe}} = \frac{\sum F_V}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{4218.05}{20}\left(1 \pm \frac{6 * 0.47}{20}\right) = 210.903(1 \pm 0.141)$$

$$\text{Therefore, } \sigma_{toe} = 240.64\frac{KN}{m^2} = 240640\frac{N}{m^2} \text{ and } \sigma_{heel} = 181.166\frac{KN}{m^2} = 181166\frac{N}{m^2}$$

Factor of safety against overturning:

$$FS_{overturning} = \frac{\sum M_R}{\sum M_O} = \frac{75562.94}{35333.78} = 2.13 > 2 \text{ safe..}$$

Factor of safety against sliding:

$$FS_{sliding} = \frac{\mu \sum F_V}{\sum F_H} = \frac{0.7 * 4218.08}{1883.52} = 1.57 > 1.5 \text{ Safe.}$$

Thus, the gravity dam is safe against overturning and sliding. Further, it has to be assured on bearing capacity and seepage.

Similarly, stresses on heel and toe is found at full reservoir conditions for 20m height dam and 10m height dam. The seismic loading and flood condition are not considered to determine vertical stress on heel and toe. The free board for 20m and 10m height dam are 6m and 2m respectively.

Table 4. 2: Stress on heel and toe of dam at varying height

S.N.	**Dam Height (m)**	**Base Width of Dam (m)**	**Drainage Gallery**	**FOS (Sliding)**	**FOS (Overturning)**	$\boldsymbol{\sigma_{heel}}$ **(N/m²)**	$\boldsymbol{\sigma_{toe}}$ **(N/m²)**
1.	27	20	6m from heel	1.57	2.13	181166	240640 (4m tail water)
2.	20	18	6m from heel	1.9	2.18	14820	235480 (no tail water)
3.	10	12	6m from heel	2.5	2.8	86640	122880 (no tail water)

Dam is modelled using finite difference method software. The Mohr-coulomb model is chosen for analysis. The value of bulk modulus of elasticity, shear modulus of elasticity and density for different percentage fine fraction is find out from equations derived in methodology chapter. There was no presence of clay content in the deposits. The dilation and tension effect are neglected. Friction varies with respect to fine fraction. The modelling parameters used in FDM software are tabulated below.

Table 4. 3: Mechanical Parameters used in Finite difference Method for Different Percentage Fine Fraction used on Modelling

S.N.	**Percentage fine Fraction (%)**	**Bulk Modulus (x 10^7 N/m^2)**	**Shear Modulus (x 10^7 N/m^2)**	**Density (Kg/m^3)**	**Cohesion (C)**	**Dilation**	**Tension**
1.	80	13	5.5	1359.86	0	0	0
2.	60	58	13.93	1485.26	0	0	0
3.	50	86.2	19.13	1562	0	0	0
4.	40	118.26	24.995	1628.9	0	0	0
5.	35	135.7	28.17	1654.16	0	0	0
6.	30	154.16	31.5	1671.56	0	0	0
7.	25	173.6	35	1679.3	0	0	0
8.	20	193.9	38.7	1675.6	0	0	0

4.3 Modelling of gravity dam at Linear Distribution of alluvial deposits

The four different modelling cases performed in linear homogenous distribution of material are as follows:

(a) Linear distribution of homogenous material at 80, 60 and 40 percentage fine fraction.

(b) Linear distribution of homogenous material at 50, 40 and 35 percentage fine fraction.

(c) Linear distribution of homogenous material at 30, 25 and 20 percentage fine fraction.

(d) Linear distribution of homogenous material at 20 percentage fine fraction.

4.3.1 Linear Variation of Material 80, 60 and 40 Percentage

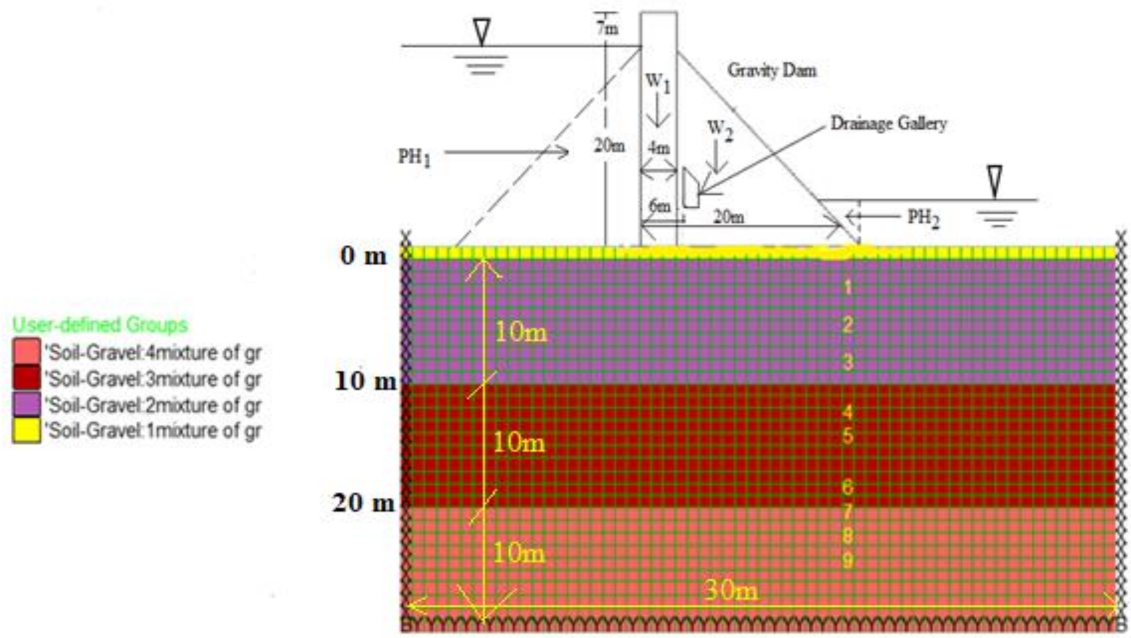

Figure 4. 2: Model of dam representing layer of 80,60 and 40 percentage fine fraction

Figure 4.2 illustrates the alluvial deposits having three layers of soil. From the definition mentioned above in methodology chapters, alluvial deposits consist of fine fraction and coarse fraction. The upper layer shown by purple colour consists of 80 percentage fine fraction and remaining coarse fraction, middle layer highlighted by red colours consists 60 percentage fine fraction and remaining coarse fraction and bottom layer consists of 40 percentage fine fraction and remaining coarse fraction. The width and depth of foundation material is taken as 60m and 30m respectively for modelling. A (1x1) m mesh is generated for the analysis. A gravity dam as shown in figure 4.1 of 20m base width and 27m height is put in middle as shown in figure 4.2. Since stress on toe is greater than stress on heel, due to this dam exerts a trapezoidal load on the foundation. Analysis is done for full reservoir conditions only. To transform load from dam to the foundation material a 1m thick stiff plate is put in model. The nine number of points (1,2,3,4,5,6,7,8,9) are allocated below toe of dam which are at the depth of 3m, 6m, 9m, 13m, 15m, 21m, 23m, and 19m respectively from surface. The deformation histories at those points are plotted at different time steps.

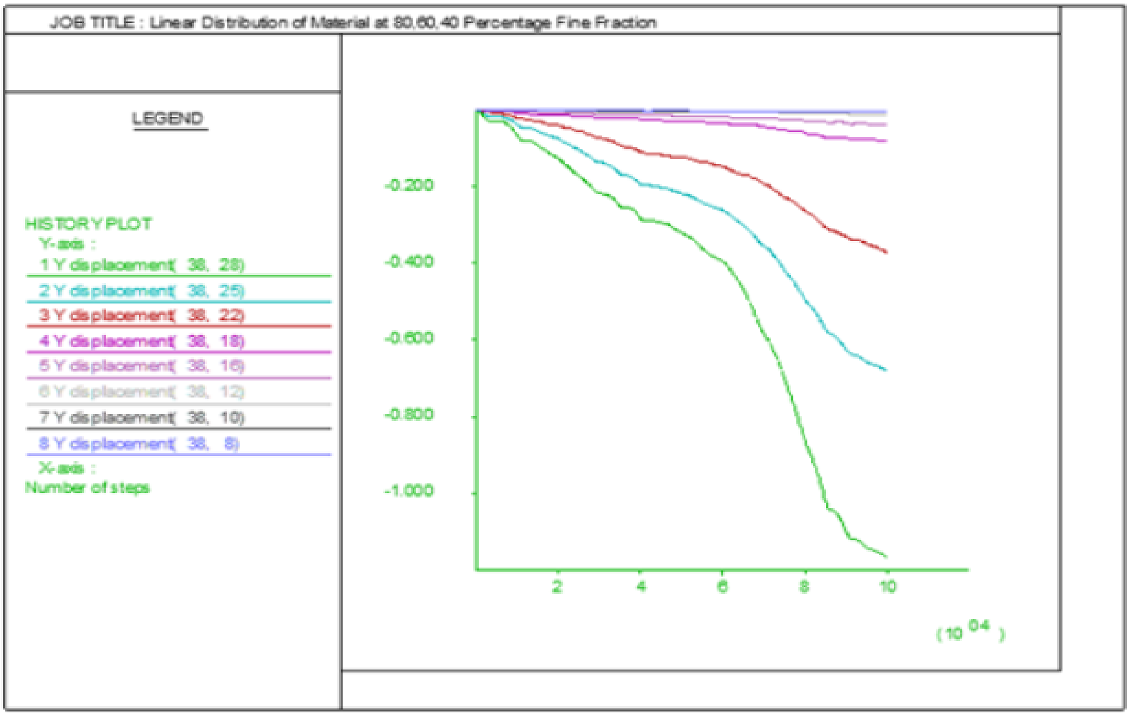

Figure 4. 3: Deformation Histories of Dam at 8 different points

The deformation in the subbase of dam is found to be more just below the toe due to more vertical stress at toe. The deformation histories at 9 different points below the toe is found out. The maximum deformation is found to be slightly more than 1m just below the toe of dam at 10^5 timesteps. One metre deformation on foundation is regarded excessively high. The bearing capacity of dam foundation is not enough to withstand the dam load. Ground improvement is required on it to strengthen the foundation material.

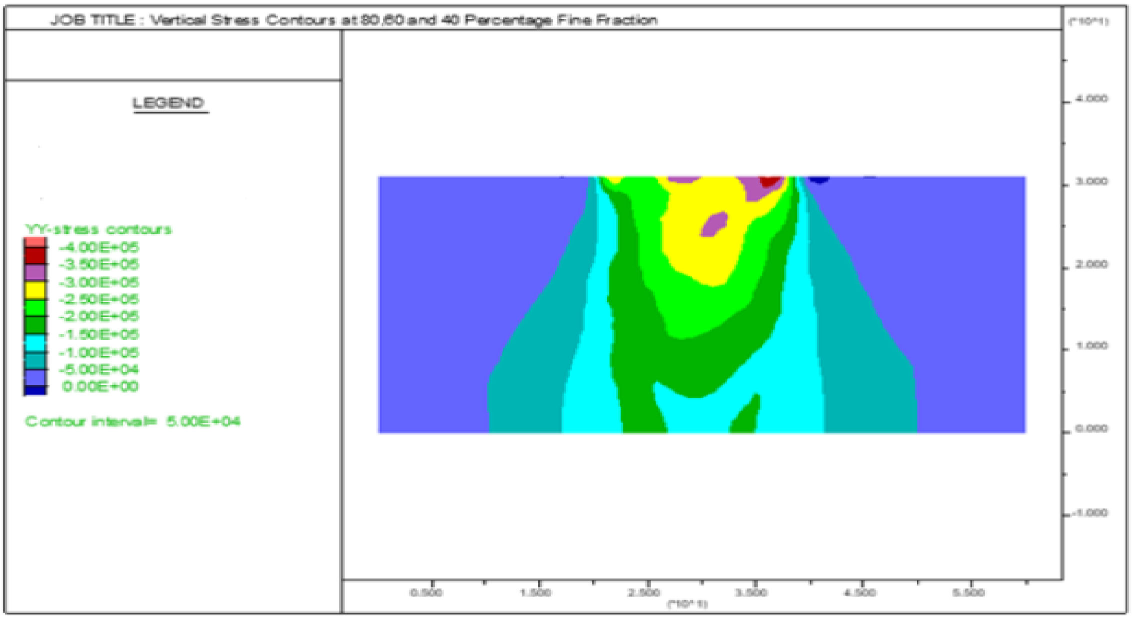

Figure 4. 4: Vertical Stress Contours at layer of 80, 60 and 40 Percentage Fine Fraction

The vertical stress contours are plotted as shown in figure above. The maximum vertical stress is found just below the toe of dam. The vertical stress decrease with increase in depth as shown in figure above by stress contours diagram. The maximum vertical stress occurs just below the toe of dam which is 3.5 x $10^5\ \frac{\mathrm{N}}{\mathrm{m}^2}$ and 3 x $10^5\ \frac{\mathrm{N}}{\mathrm{m}^2}$. The value of vertical stress contours is not uniform.

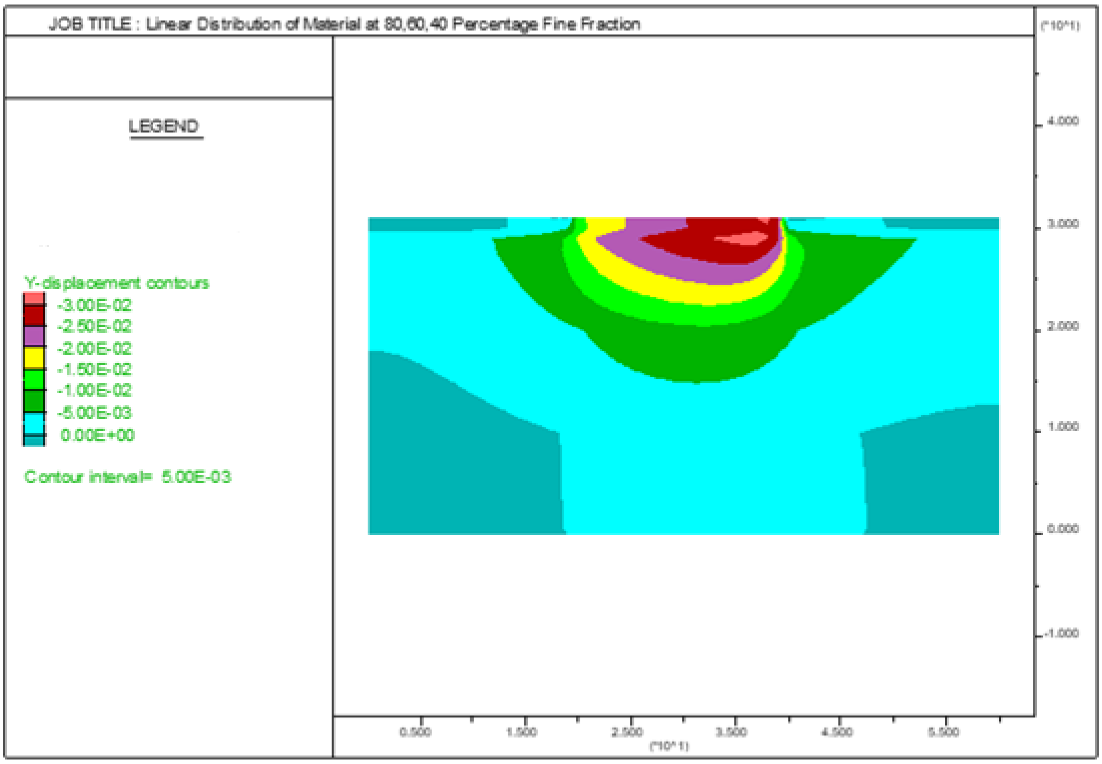

Figure 4. 5: Displacement Contours of Dam at a layer of 80,60 and 40 Percentage Fine Fraction

The maximum vertical displacement of dam is found at upper portion of dam just below the toe of dam. The vertical displacement of dam foundation also decreases with increase in depth of foundation. The vertical deformation due to dam dissipates to a negligible value below the 17m depth of foundation. The maximum vertical displacement is found to be 30mm and 25mm just below the plate of dam.

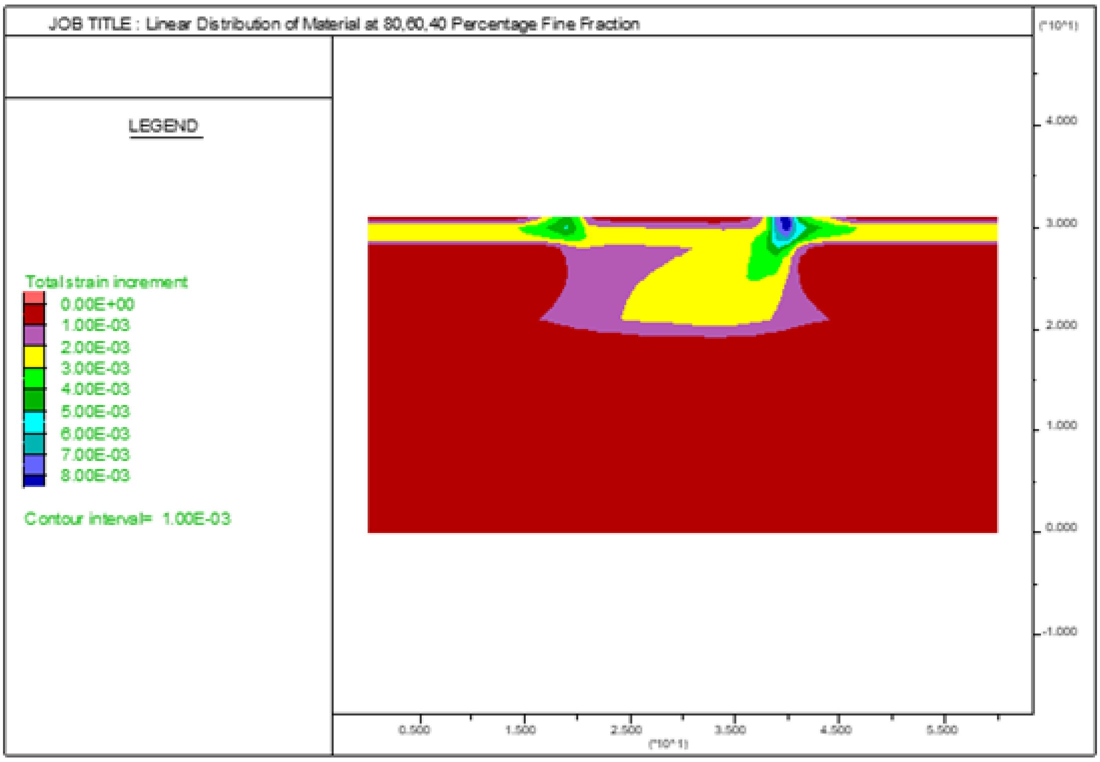

Figure 4. 6: Total strain increment at layer of 80, 6 0 and 40 Percentage Fine Fraction

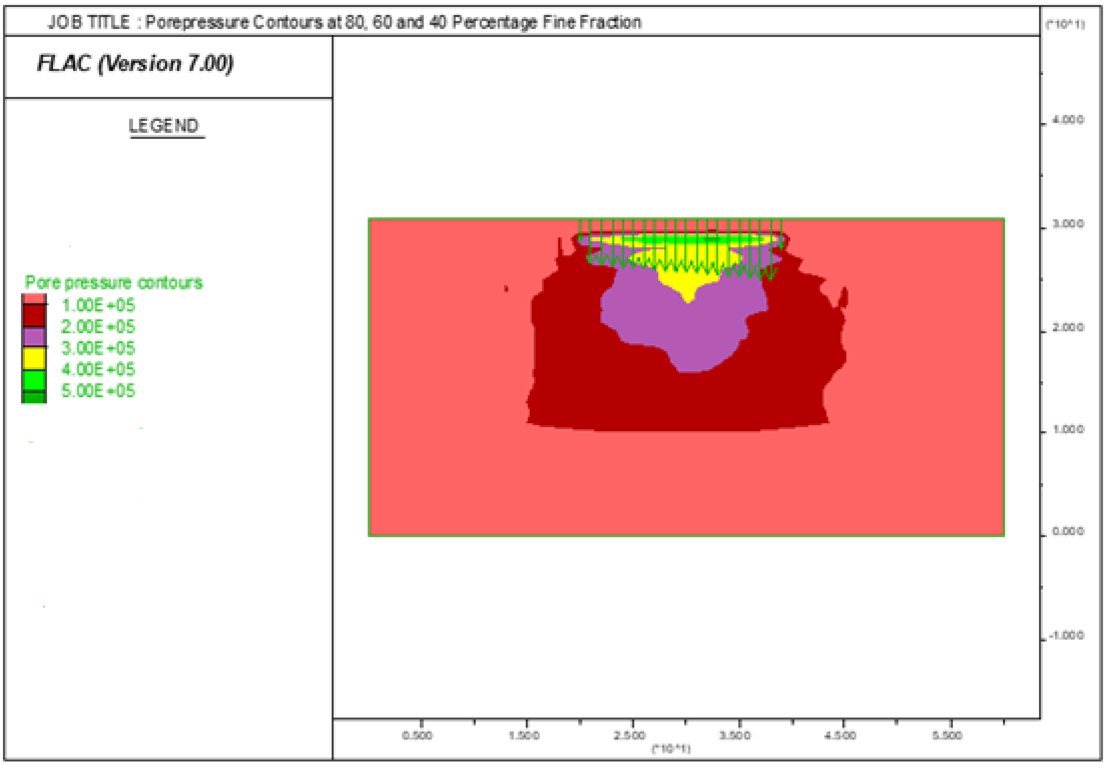

Figure 4. 7: Pore pressure Contours at layer of 80, 60 and 40 Percentage Fine Fraction

The excess pore pressure occurs maximum at the top of dam foundation. The maximum pore pressure is found to be 4 X $10^5 \frac{N}{m^2}$ just below the toe of dam foundation.

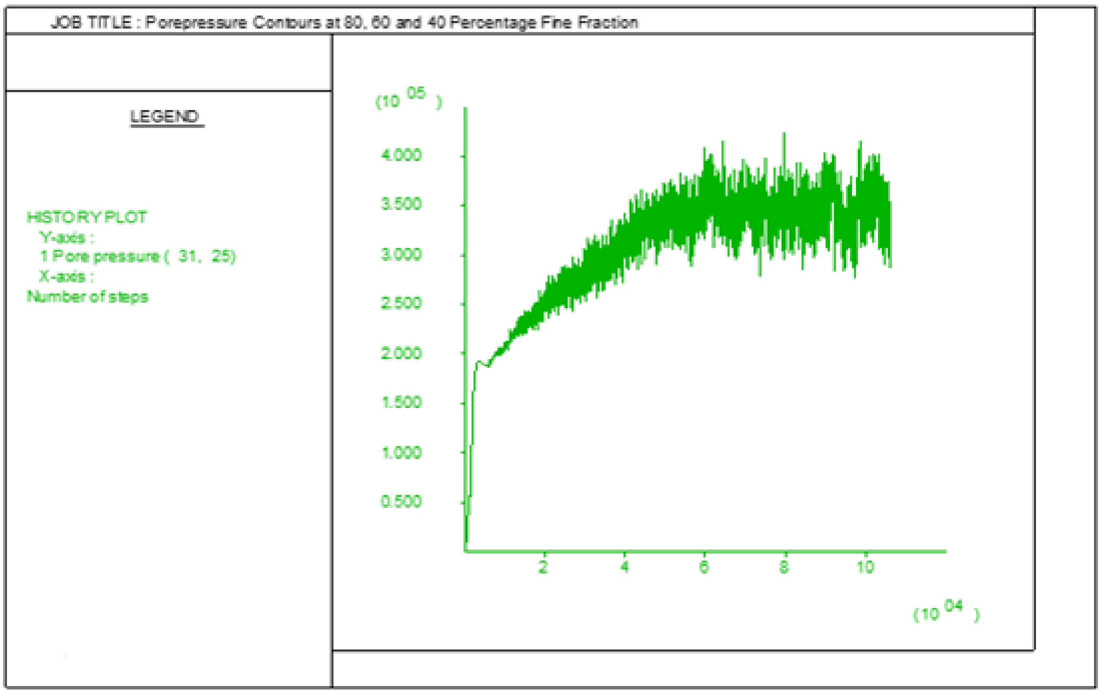

Figure 4. 8: Pore pressure histories at layer of 80, 60 and 40 Percentage Fine Fraction

The value of pore pressure histories at different time steps is plotted on the above graph. It shows that the pore pressure increases with increase in timesteps. The maximum value of pore pressure is found to be 4 X $10^5 \frac{N}{m^2}$ at points (31,25) from pore pressure histories plot.

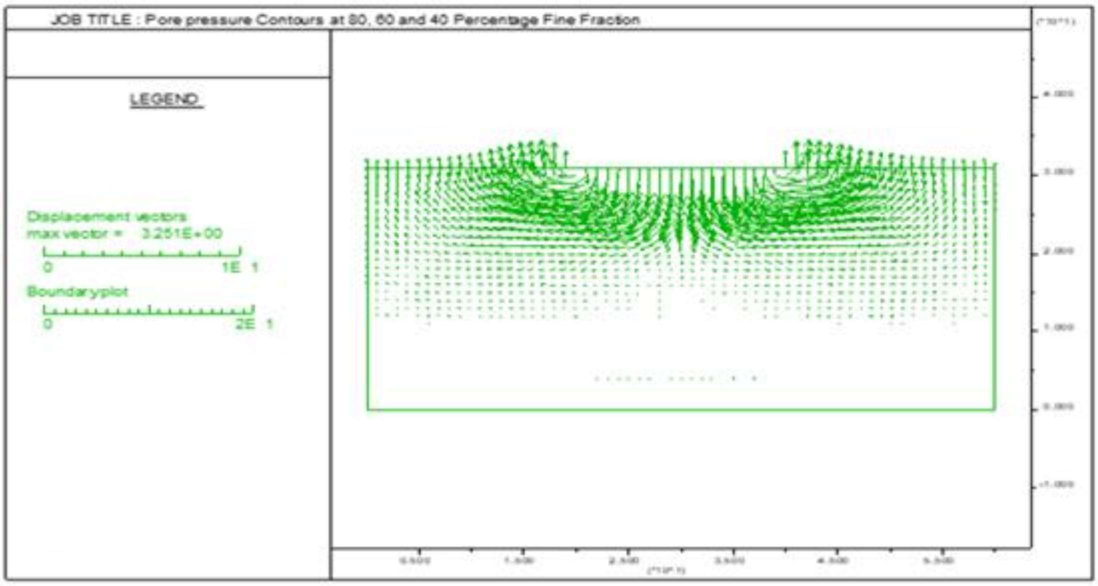

Figure 4. 9: Displacement vectors at layers of 80, 60 and 40 Percentage Fine Fraction

The above figure shows the displacement vectors. The maximum vector is found to be 3.25.

4.3.2 Linear Distribution of Material 50, 40, and 35 percentage Fine Fraction

The assumptions are alluvial deposits consist of fine fraction and coarse fraction. The upper layer consists of 50 percentage fine fraction and remaining coarse fraction, middle layer 40 percentage fine fraction and remaining coarse fraction and bottom layer 35 percentage fine fraction and remaining coarse fraction. The width and depth of base material is taken as 60m and 30m respectively for modelling. A gravity dam as shown in figure 4.1 of 20m base width and 27m height is put in middle of model as shown in figure 4.2. Dam exerts a trapezoidal load on the model. The vertical downward arrow shown in above model is a vertical stress due to dam load. To transform load from dam to the foundation material a 1m thick stiff plate is put in model.

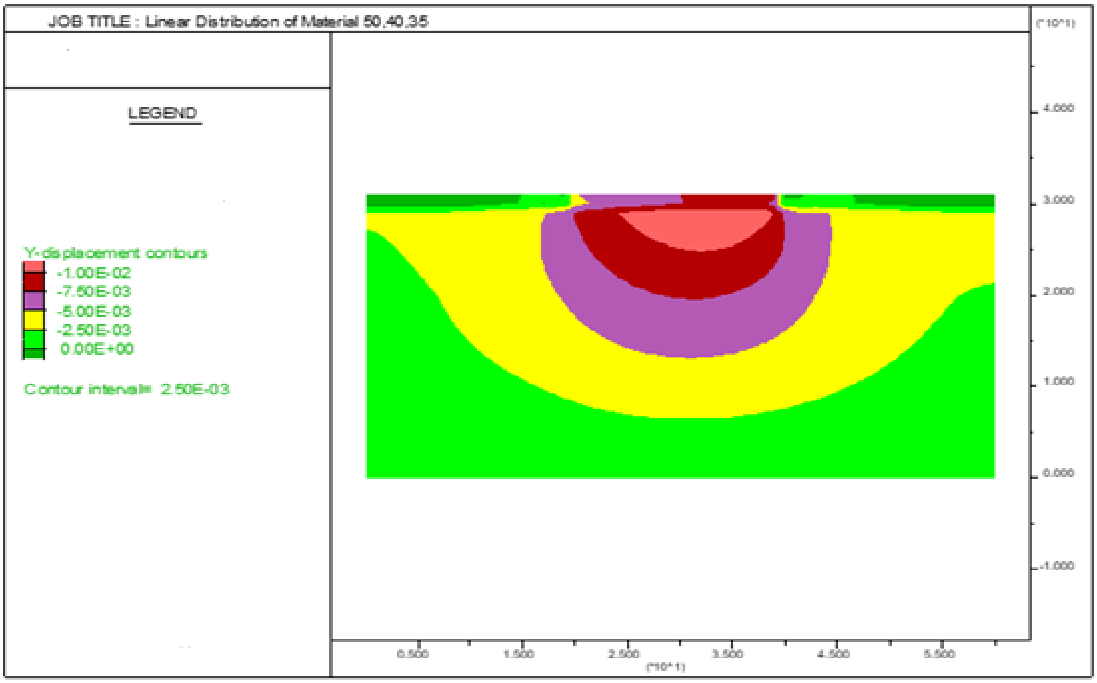

Figure 4. 10: Vertical displacement contours at 50, 40 and 35 percentage fine fraction

The vertical displacement of dam is found to be at upper portion of dam just below the toe of dam. The vertical displacement of dam foundation decreases with increase in depth of foundation. The vertical deformation due to dam dissipates to a negligible value below the 17m depth of foundation. The maximum vertical displacement is found to be 10mm and 7.5mm just below the plate of dam. The value of displacement decreases slightly after decrease in percentage fine fraction.

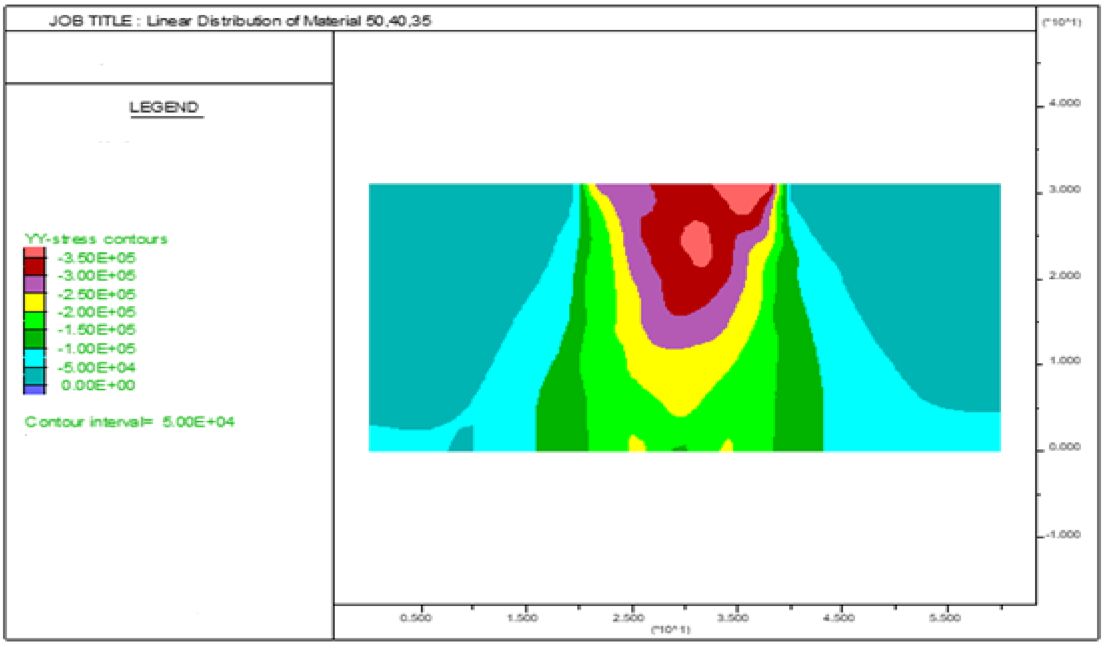

Figure 4. 11: Vertical Stress contours at 50, 40 and 35 percentage fine fraction

The vertical stress contours are plotted as shown in figure above. The maximum vertical stress is found to be just below the toe of dam. The vertical stress decrease with increase in depth as shown in figure above by stress contours diagram. The maximum vertical stress occurs just below the toe of dam which is 3.5 x 10^5 $\frac{\mathrm{N}}{\mathrm{m}^2}$ and 3 x 10^5 $\frac{\mathrm{N}}{\mathrm{m}^2}$.

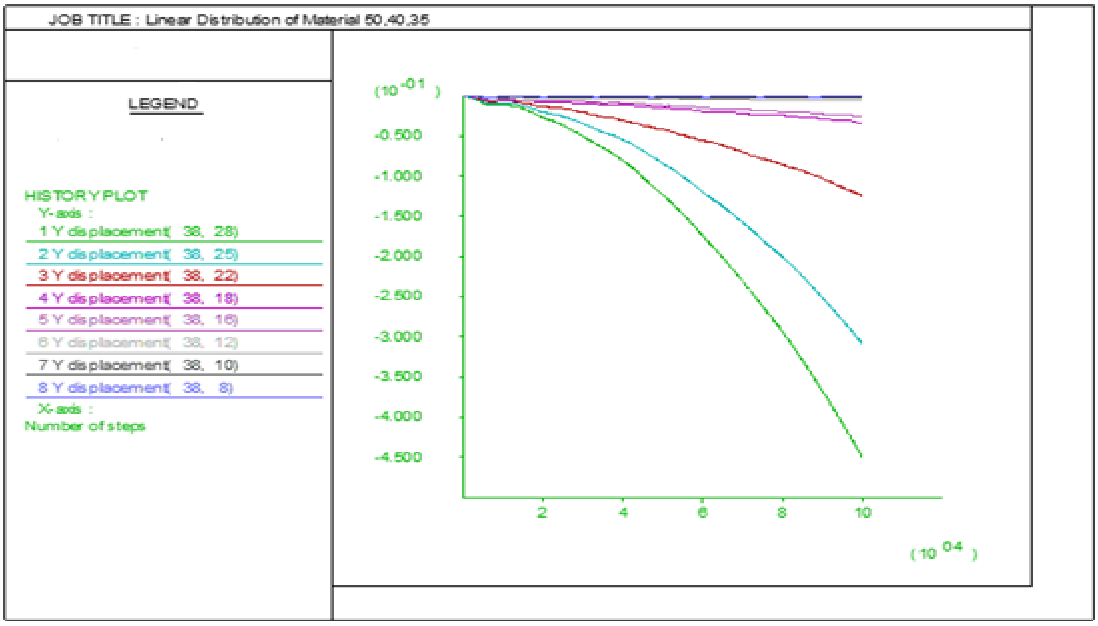

Figure 4. 12: Vertical displacement histories at 8 different points

The deformation in the subbase of dam is found to be comparatively more just below the toe due to more vertical stress at toe. The deformation histories at 8 different points below the toe is found out in above figure 4.12. The maximum deformation is found to be slightly more than 400mm just below the toe of dam. The deformation histories at different time steps shows that the deformation decreases with respect to decrease in depth.

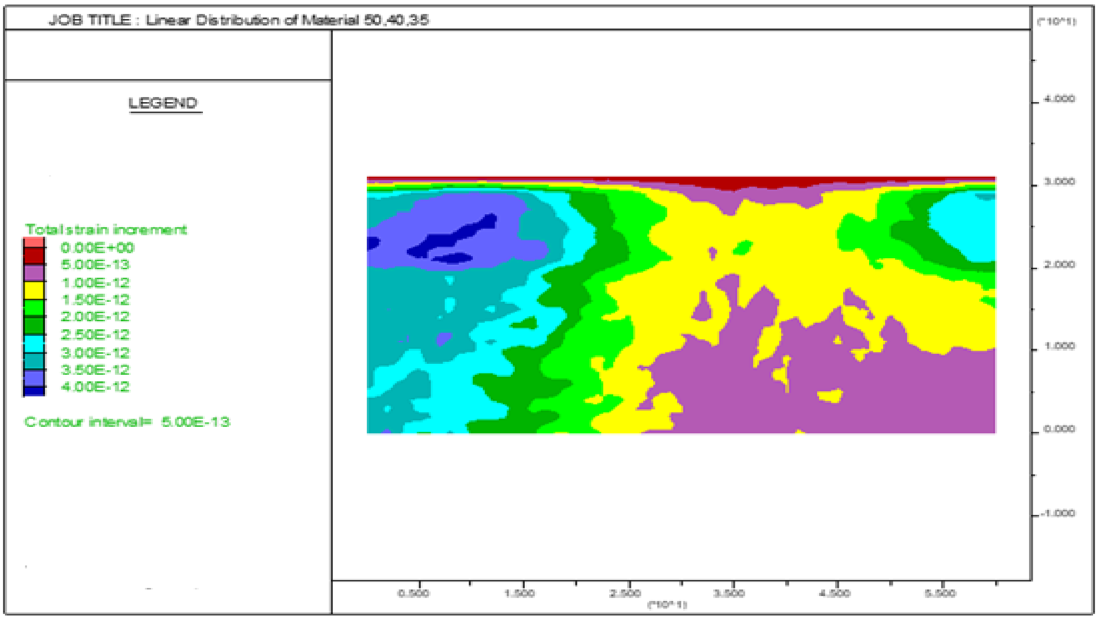

Figure 4. 13: Total strain increment at layer of 50, 40 and 35 Percentage Fine fractions

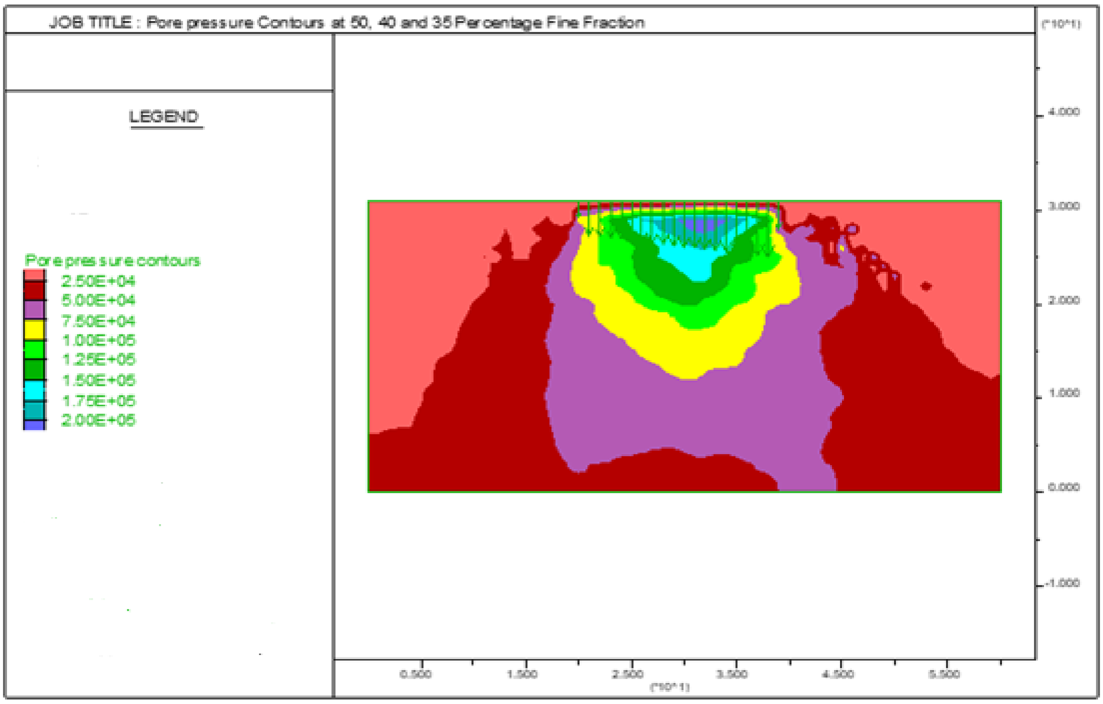

Figure 4. 14: Pore pressure contours at 50, 40 and 35 percentage fine fraction

The excess pore pressure occurs maximum at the top of dam foundation. The maximum pore pressure is found to be 2 X $10^5 \frac{N}{m^2}$ just below the toe portion of dam foundation. With increase in coarse content of foundation material the value of pore pressure decreases.

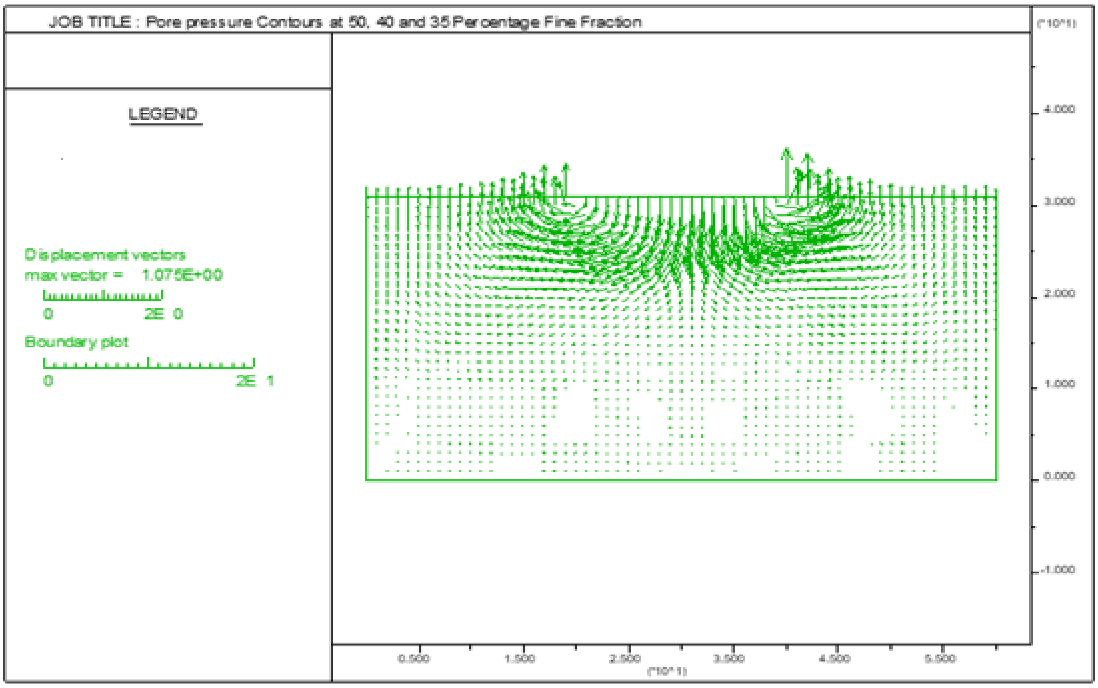

Figure 4. 15: Displacement vectors at 50, 40 and 35 Percentage fine fraction

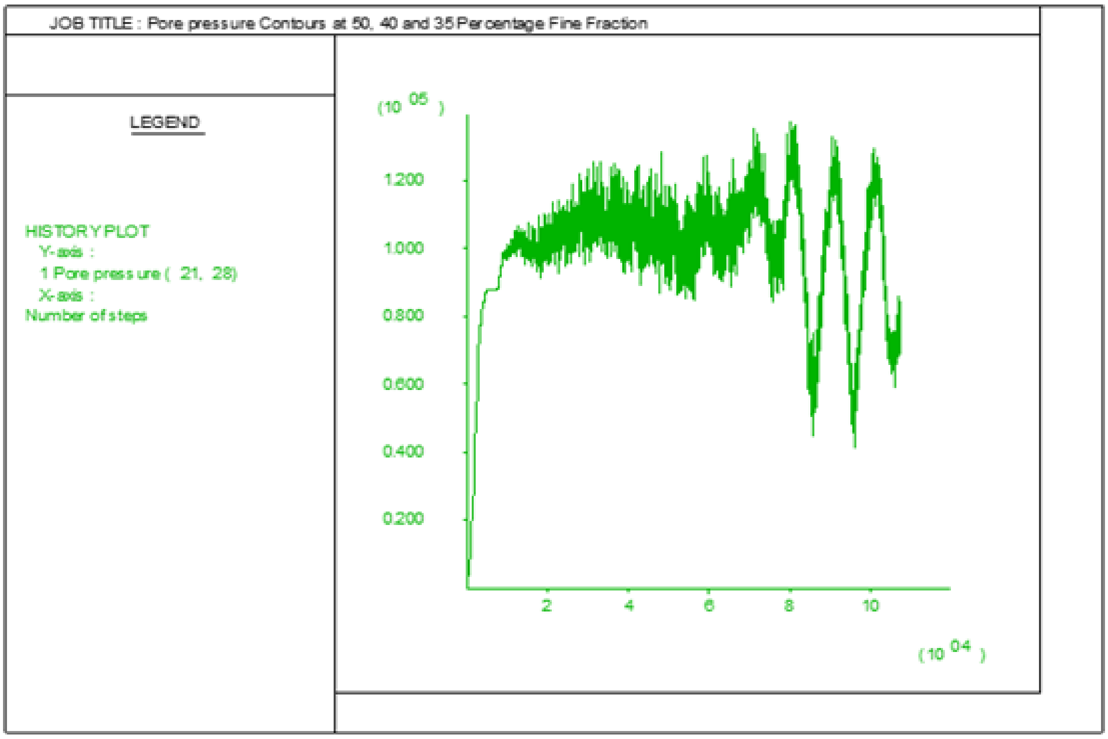

Figure 4. 16: Pore pressure histories at point (21,28) on different time steps

The value of pore pressure histories at different time steps is plotted on the above graph. It shows that the pore pressure value fluctuates with increase in timesteps. The maximum value of pore pressure is found to be 1.25 X 10^5 $\frac{\mathrm{N}}{\mathrm{m}^2}$ from pore pressure histories plot.

4.3.3 Linear Variation of Material 30, 25 and 20 Percentage Fine Fraction

The material is modelled considering linear variation of 30 percentage fine fraction, 25 percentage fine fraction and 20 percentage fine fraction. The upper layer consists of 30 percentage fine fraction and remaining coarse fraction, middle layer 25 percentage fine fraction and remaining coarse fraction and bottom layer 20 percentage fine fraction and remaining coarse fraction. The width and depth of base material is taken as 60m and 30m respectively for modelling. A gravity dam of 27m height and 20m base width is modelled as shown in figure 4.2. All other conditions are similar as discussed in 4.2.1.

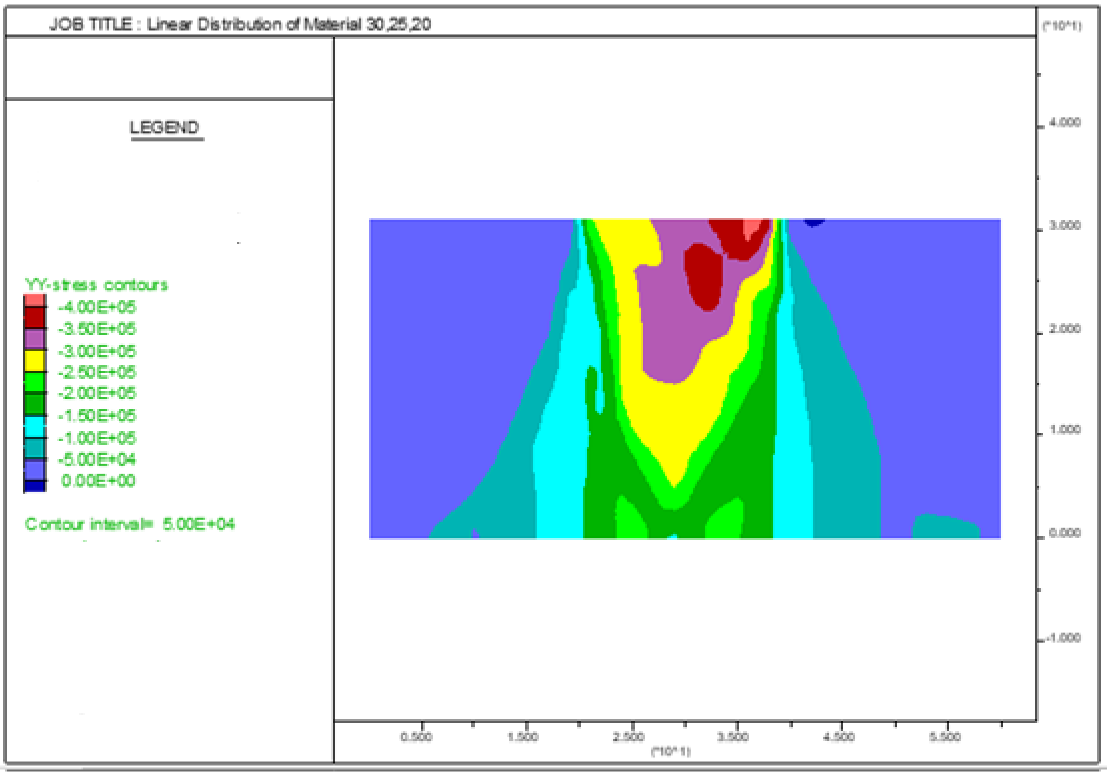

Figure 4. 17: Vertical Stress Contours at 30, 25, and 20 Percentage Fine Fraction

The vertical stress contours are plotted as shown in figure above. The maximum vertical stress is found just below the toe of dam. The vertical stress decrease with increase in depth as shown in figure above by stress contours diagram. The maximum vertical stress occurs just below the toe of dam which is 3.5 x 10^5 $\frac{\mathrm{N}}{\mathrm{m}^2}$ and 3 x 10^5 $\frac{\mathrm{N}}{\mathrm{m}^2}$. The value of maximum stress does not change due to grouting. But the stress contours change slightly due to grouting of alluvial deposits.

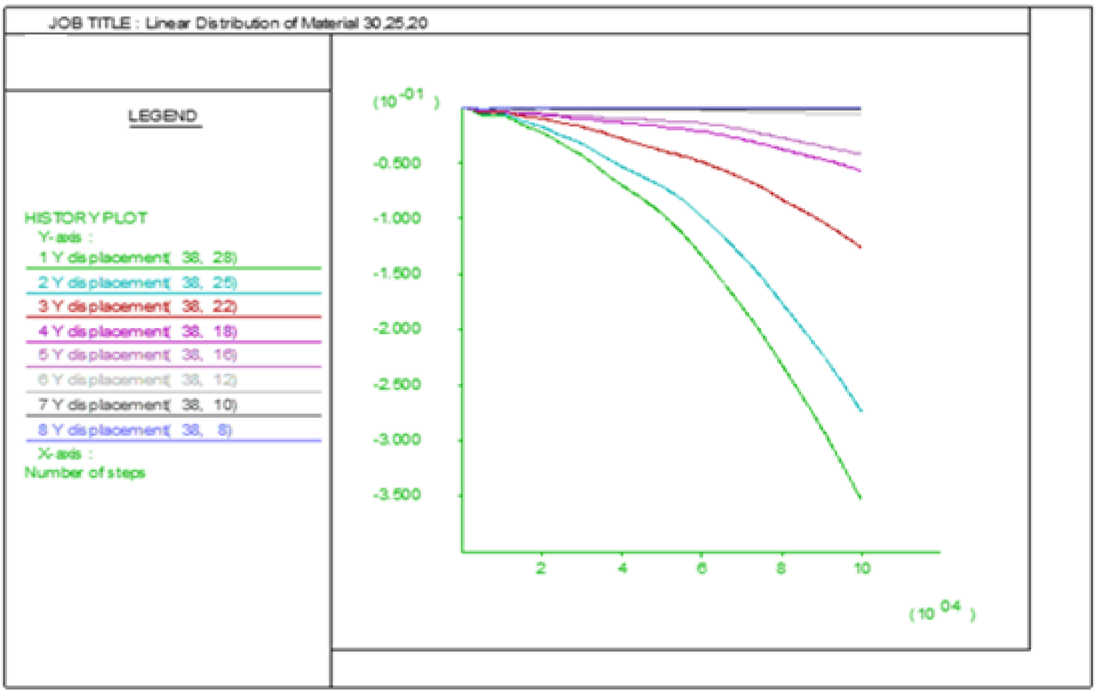

Figure 4. 18: Vertical displacement Contours at 8 different points

The deformation in the subbase of dam is found to be comparatively more just below the toe due to more vertical stress at toe. The deformation histories at 8 different points below the toe of dam is found out in above figure 4.20. The maximum deformation is found slightly more than 350mm just below the toe of dam. The deformation histories at different time steps shows that the deformation decreases with respect to decrease in depth.

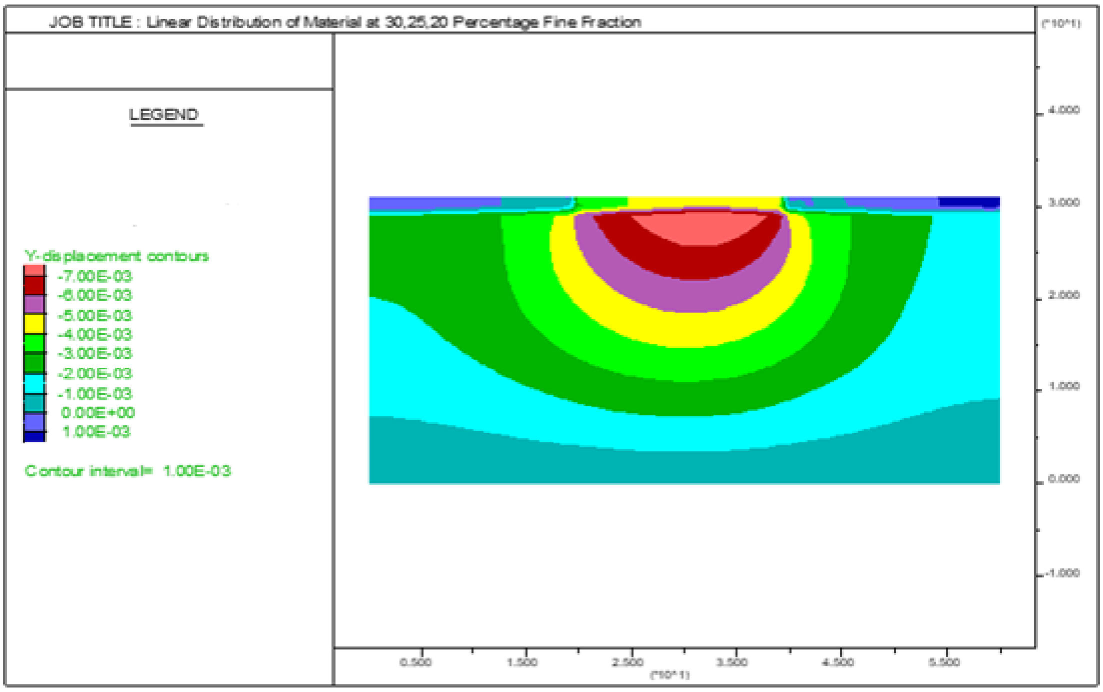

Figure 4. 19: Vertical displacement contours at 30, 25 and 20 percentage fine fraction

The maximum vertical displacement of dam is found at upper portion of dam just below the toe of dam. The vertical displacement of dam foundation decreases with increase in depth of foundation. The vertical deformation of dam below the 25m depth is almost negligible. The maximum vertical displacement is found 7mm and 6mm just below the plate of dam. Comparing figure 4.21 with figure 4.11, the vertical displacement of dam decreases with increase in percentage coarse fraction which mean decrease in fine fraction.

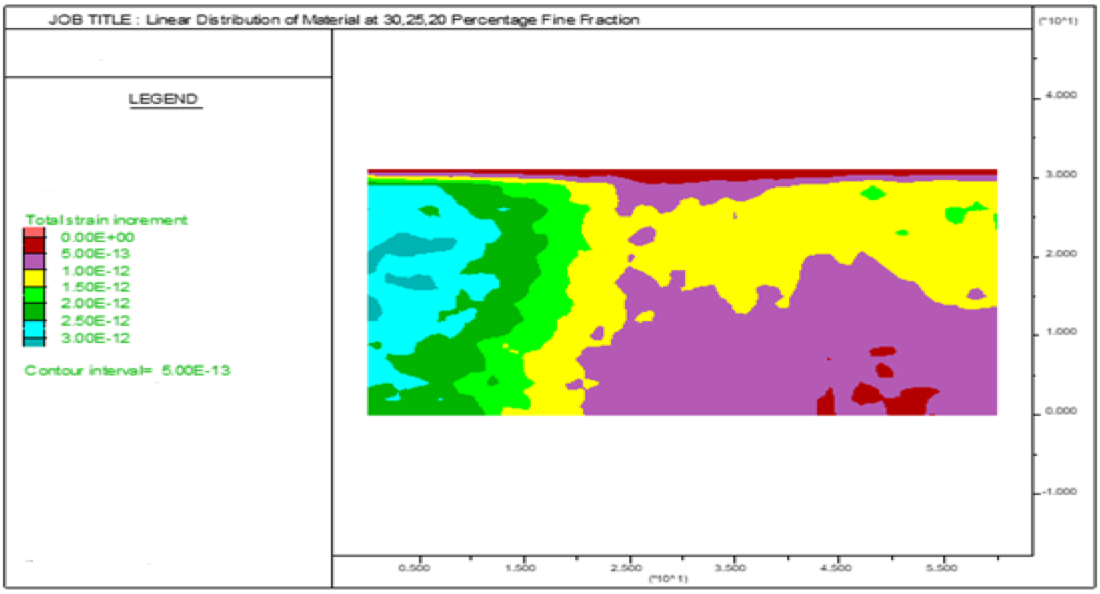

Figure 4. 20: Total Strain increment at 35, 25, and 20 percentage fine fraction

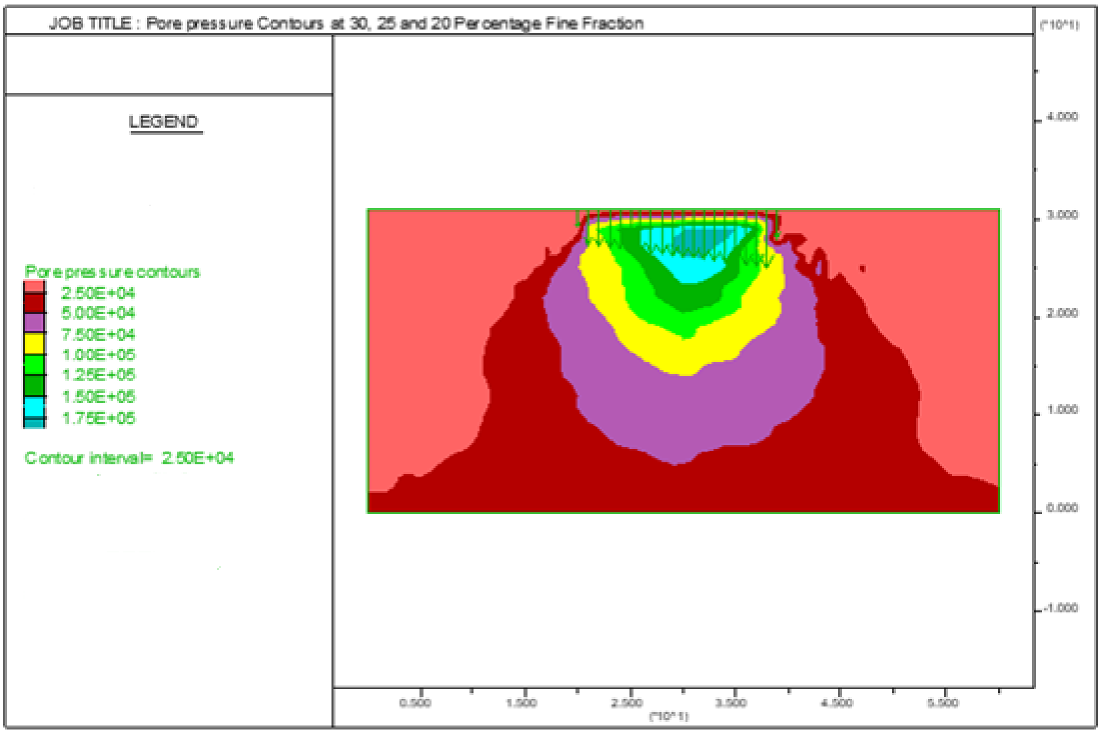

Figure 4. 21: Pore pressure contours at 30, 25 and 20 percentage fine fraction

The excess pore pressure occurs maximum at the top of dam foundation. The maximum pore pressure is 1.75 X $10^5 \frac{N}{m^2}$ just below the toe portion of dam foundation. With increase in coarse content of foundation material the value of pore pressure decreases. This can be seen by comparing fig 4.23 with 4.15. The value of excess pore pressure decreases with decrease in depth as illustrated in figure 4.23.

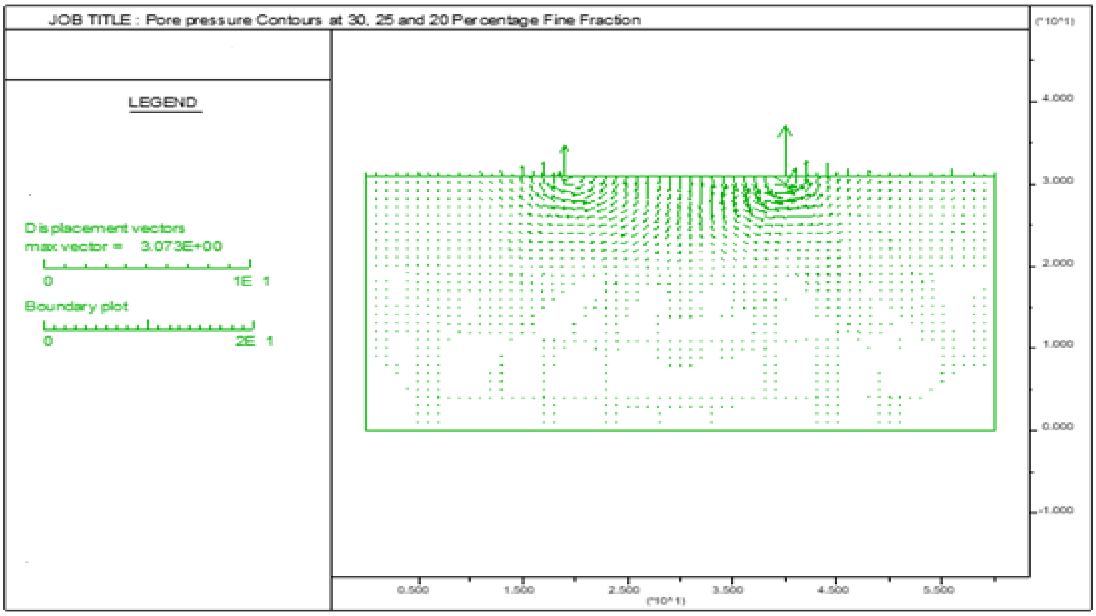

Figure 4. 22: Displacement Vectors at 30, 25, and 20 percentage fine fraction

The above figure shows the displacement vectors at 30, 25 and 20 percentage fine fraction.

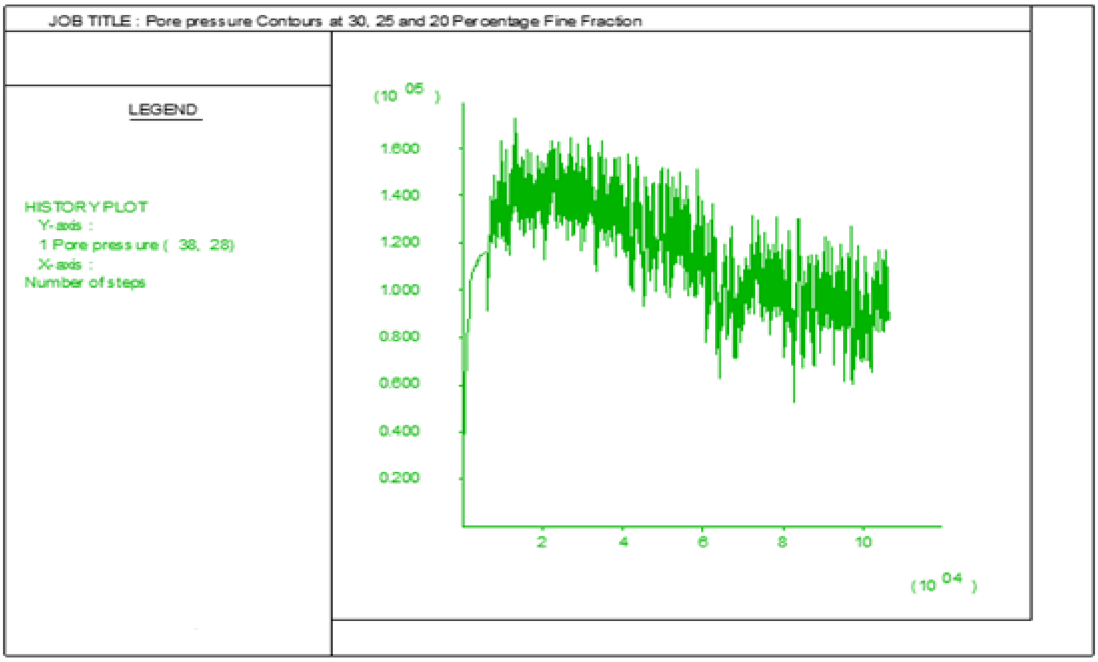

Figure 4. 23: Pore pressure Contour on point (38, 28) at 30, 25 and 20 percentage fine fraction

From above figure, the maximum value of pore pressure is found to be 1.6 X $10^5 \frac{\mathrm{N}}{\mathrm{m}^2}$. In figure 4.17, the value of maximum pore pressure is found to be 1.25 X $10^5 \frac{\mathrm{N}}{\mathrm{m}^2}$ which is less than value from figure 4.25. This is due to the fact that, one point is below the heel of dam and another is below the toe.

4.3.4 Linear Distribution of Material at 20 Percentage Fine Fraction

Above figure, 4.26 shows model of dam containing uniform distribution of 20 percentage fine fraction and remaining coarse fraction as a single layer.

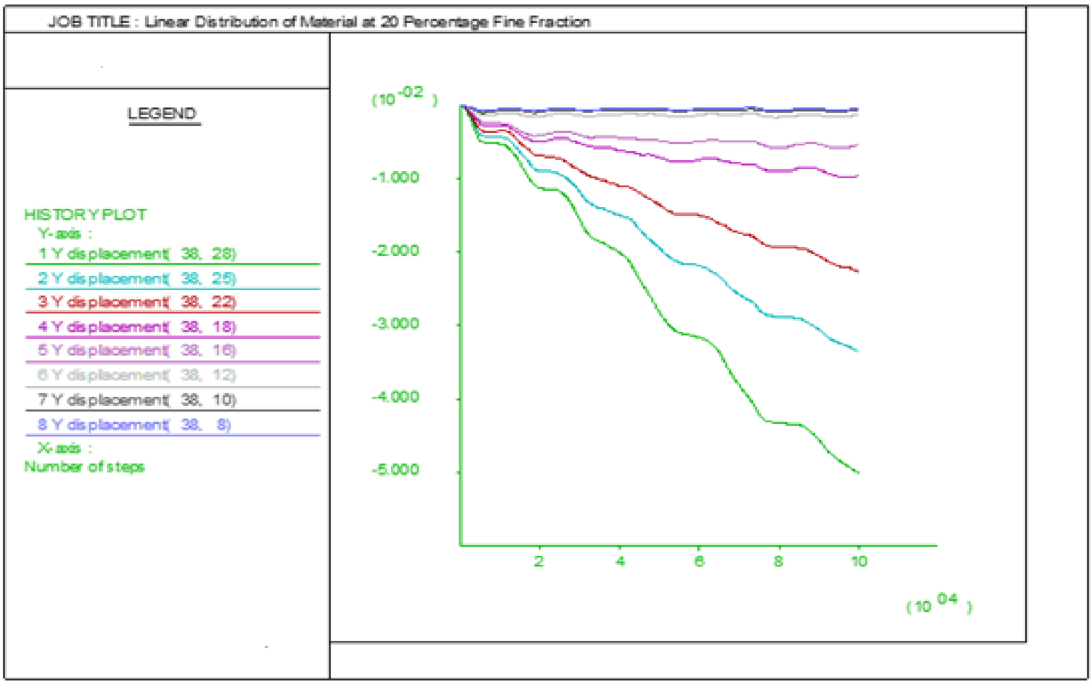

Figure 4. 24: Displacement histories at 8 different points below toe of dam

The deformation in the subbase of dam is comparatively more just below the toe due to more vertical stress at toe. The deformation histories at 8 different points below the toe of dam is found out in above figure 4.27. The maximum deformation is found slightly more than 50mm just below the toe of dam. The deformation histories at different time steps shows that the deformation decreases with respect to decrease in depth.

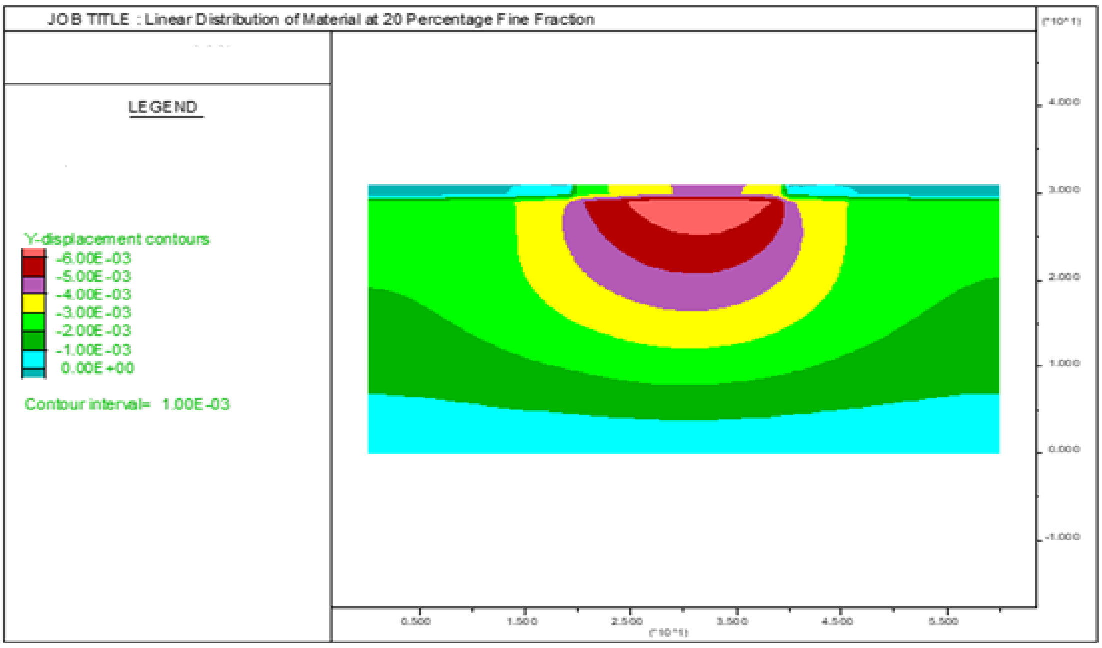

Figure 4. 25: Vertical displacement of dam at 20 percentage fine fraction

The maximum vertical displacement of dam is found to be 6mm just below the dam foundation. The vertical deformation on dam foundation decreases proportionally with increase in depth of foundation.

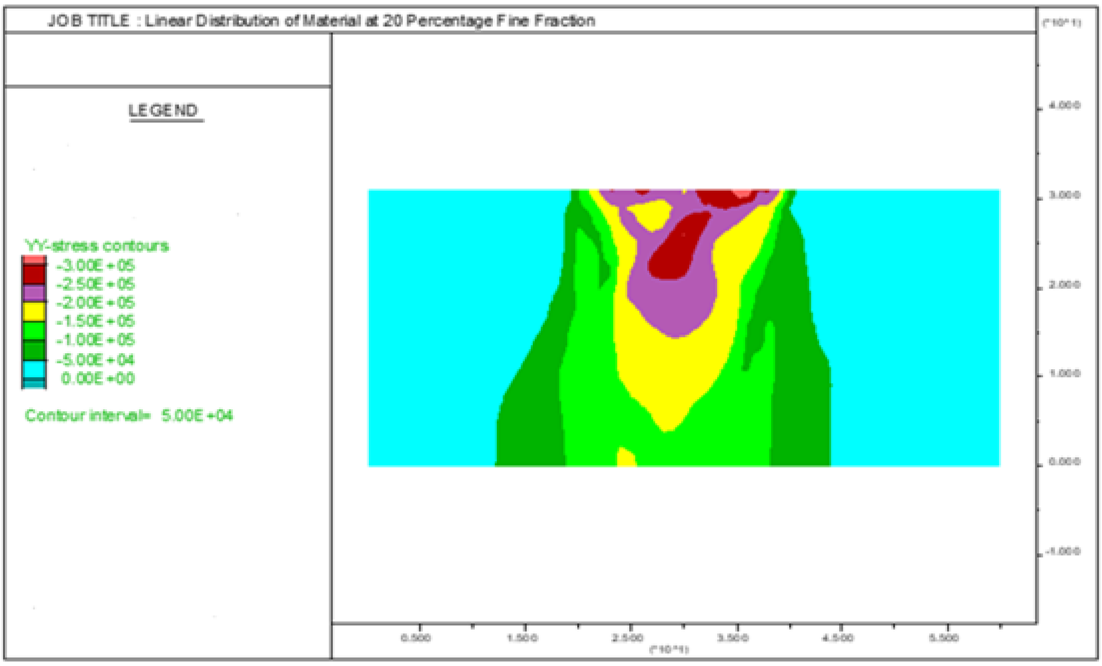

Figure 4.26: Vertical Stress Contours at 20 percentage fine fraction

From above figure 4.29, it is clear that with increase in grouting there would not be much change in vertical stress.

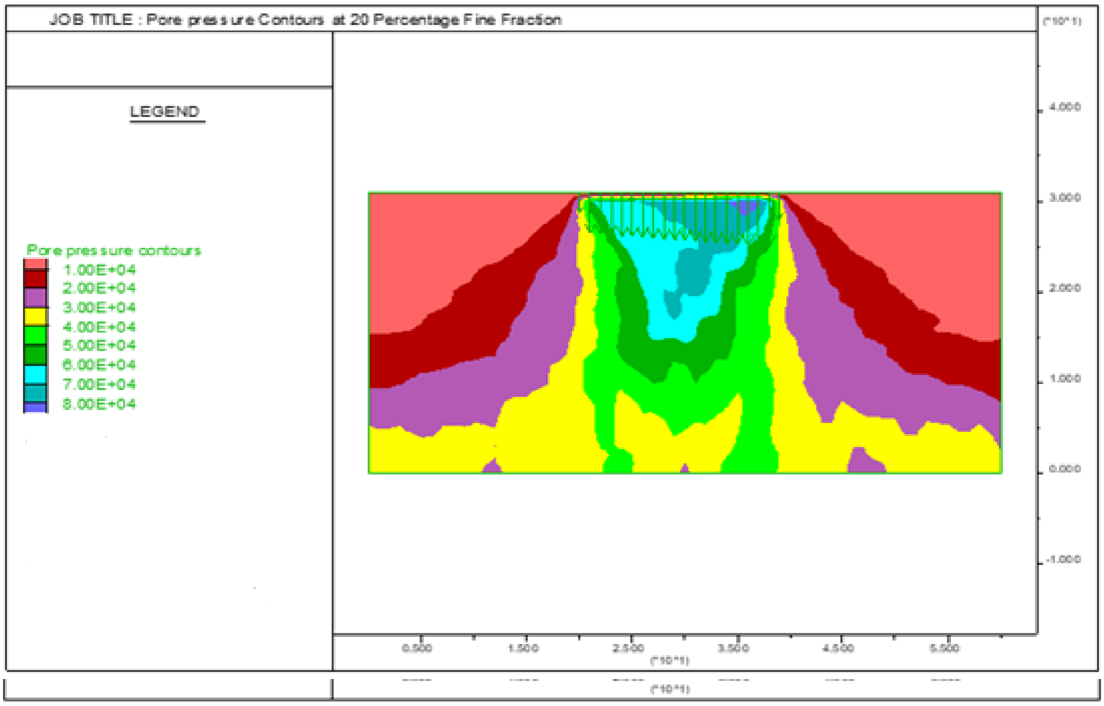

Figure 4.27: Pore Pressure Contours at 20 Percentage fine fraction

The excess pore pressure occurs maximum at the top of dam foundation. The maximum pore pressure is found to be 7 X $10^4 \frac{N}{m^2}$ just below the toe portion of dam foundation. With increase in coarse content of foundation material the value of pore pressure decreases. This can be seen by comparing fig 4.23 with 4.30. The value of excess pore pressure decreases with decrease in depth.

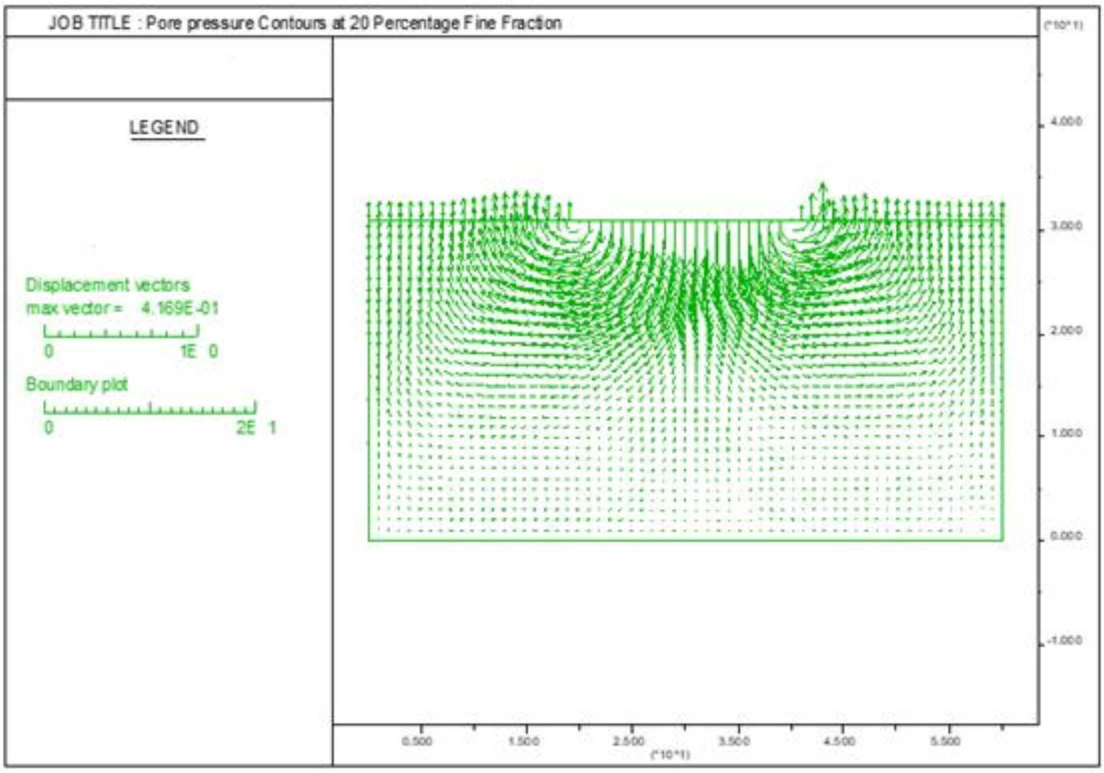

Figure 4.28: Displacement vectors at 20 percentage fine fraction

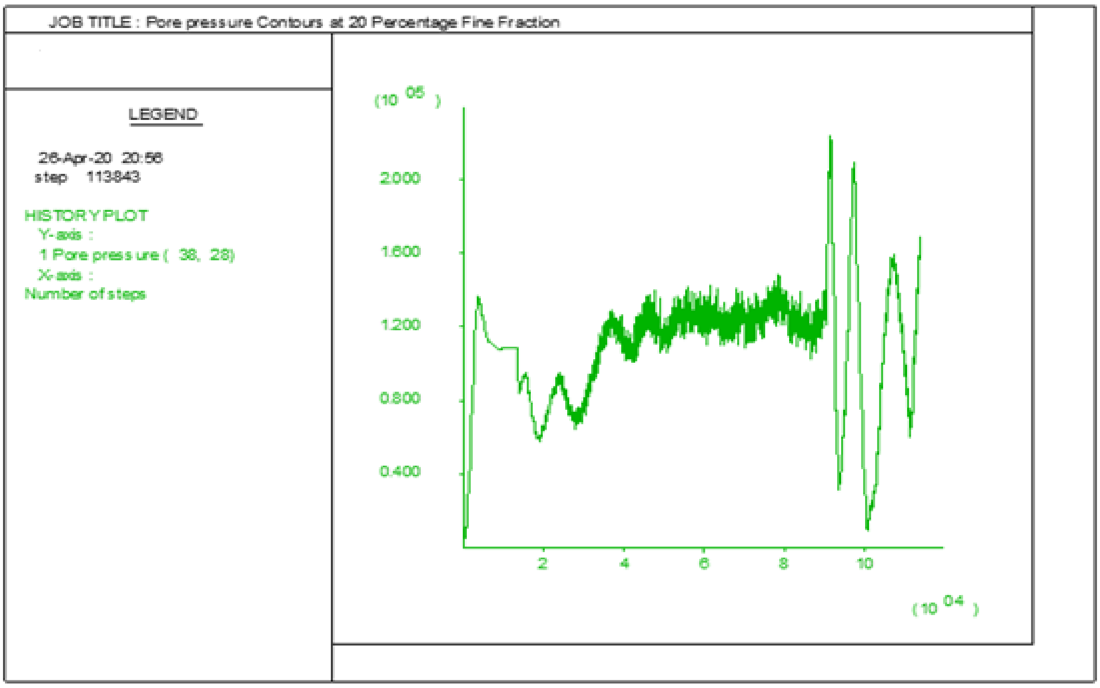

Figure 4.29: Pore pressure contours displacement histories at 20 percentage fine fraction

From above figure, the maximum value of pore pressure is found to be 1.4 X $10^5 \frac{N}{m^2}$.

4.3.5 Analysis of Deformation histories at grid location (38,28) i.e. at 3m depth

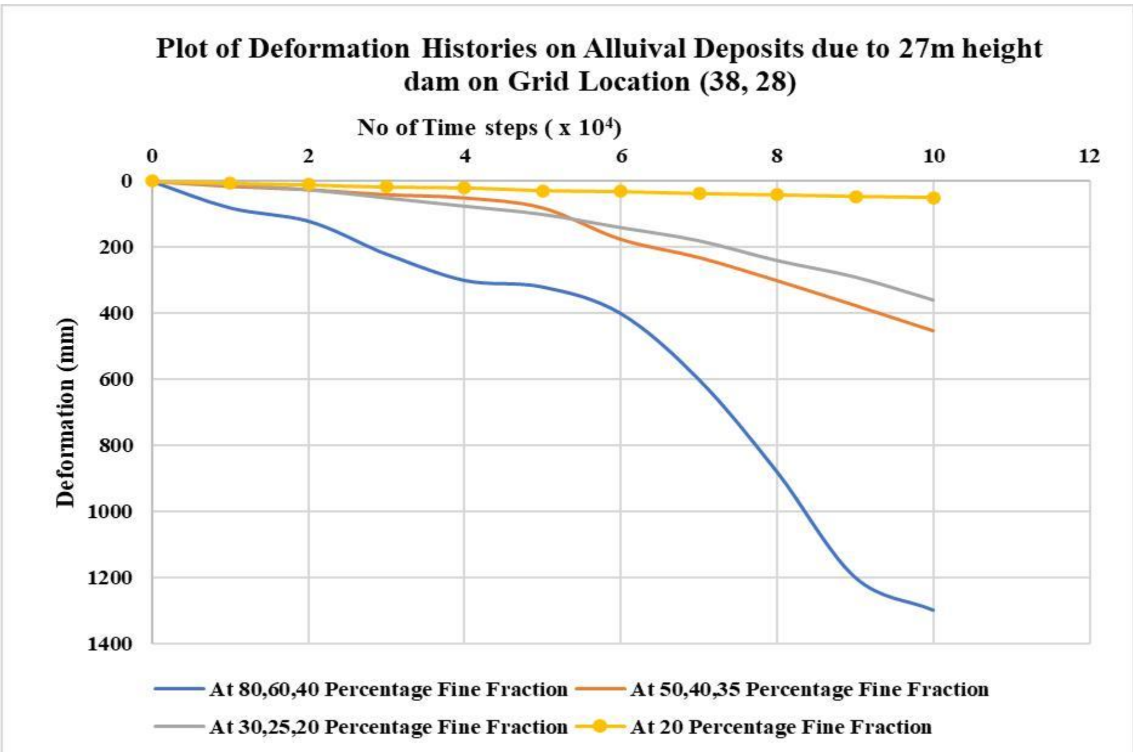

Figure 4. 30**:** Plot of deformation histories at location (38,28) containing three different layers of Alluvial Deposits

The alluvial deposit is distributed linearly in three layers with each layer of 10m thickness having different percentage fine fraction. The material is distributed linearly from top to bottom subsequently as shown in legend of above figure. Gravity dam of 27m at full reservoir conditions exerts a trapezoidal load of 1181166 KN/m^2 to 240640 KN/m^2 on heel and toe of dam respectively. By numerically modelling the dam using finite difference method above graph is obtained at grid point location (38,28). Above graph illustrates that, the maximum deformation occurs just below toe of dam at grid point (38, 28) which is around 1300 mm at 80,60,40 percentage fine fraction which is 3m below surface. The deformation reduces to around 400 mm and 300mm respectively at 50, 40, 35 percentage fine fraction and 30, 25, 20 percentage fine fraction at same location. At 20 percentage fine fraction, the maximum deformation is found around 50mm.

4.3.6. Analysis of Deformation histories at Grid location of (38,25) i.e. at 6m depth

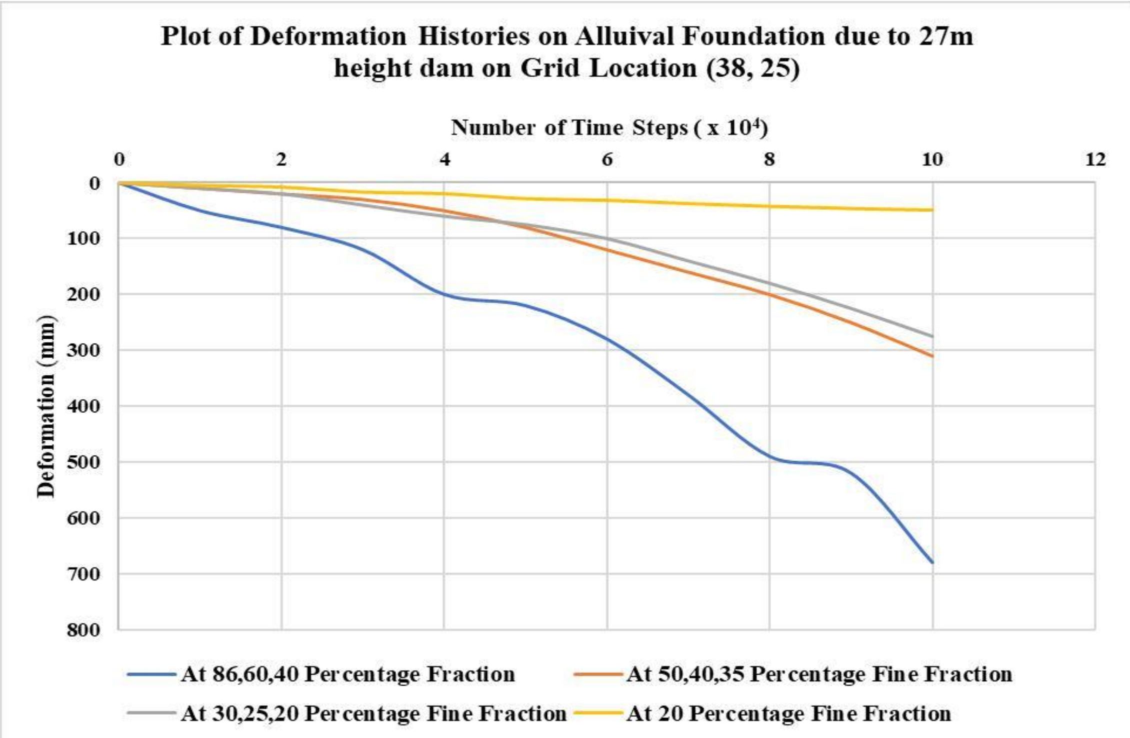

Figure 4. 31**:** Plot of deformation histories at location (38,25) containing three different layers of Alluvial Deposits

By numerically modelling the dam using finite difference method above graph is obtained at grid point location (38,25). Above graph illustrates that, the deformation of around 675mm occurs at grid point (38, 25) at linear distribution of 80, 60, 40 percentage fine fraction. It is 5m below the surface. The deformation reduces around 300 mm and 275mm respectively at 50, 40, 35 and 30, 25, 20 percentage fine fraction at same location. At linear distribution of 20 percentage fine fraction in 30m depth, maximum deformation is found around 34mm.

4.2.7. Analysis of Deformation histories at Grid location of (38, 22) i.e. at 9m depth

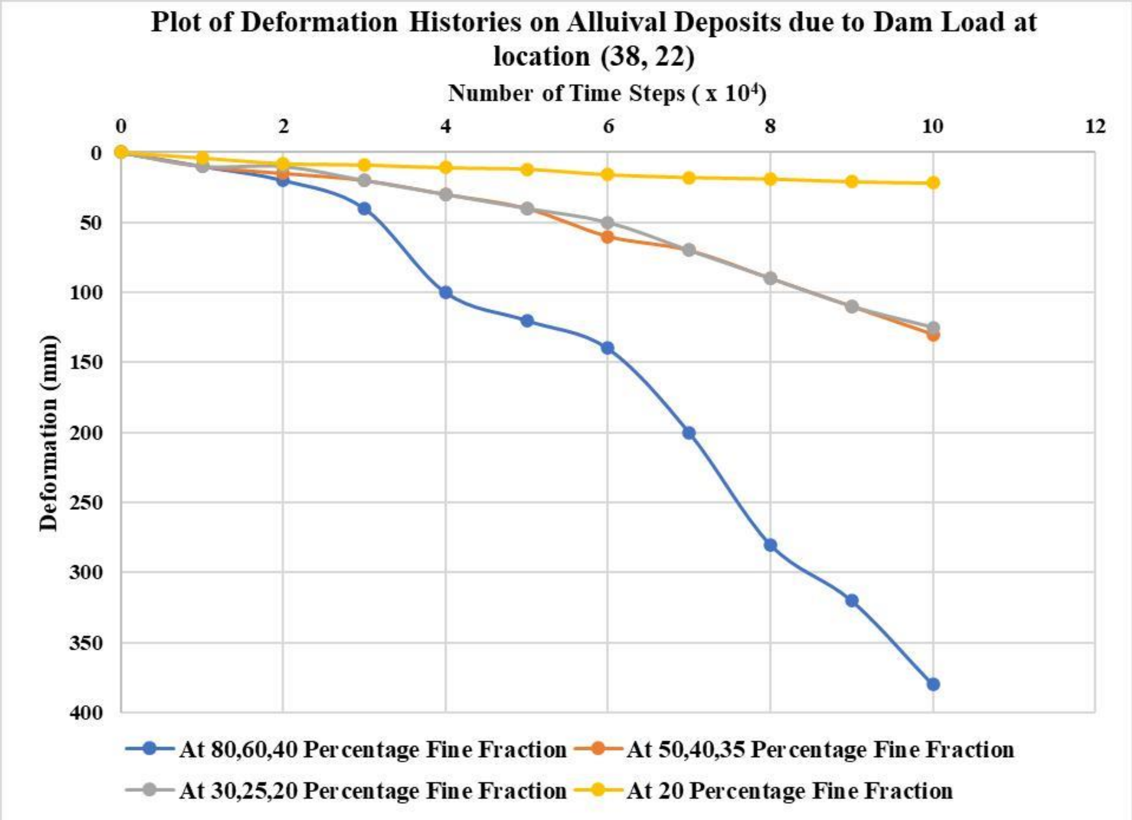

Figure 4. 32: Plot of deformation histories at location (38,22) containing three different layers of Alluvial Deposits

By numerically modelling the dam using finite difference method software above graph is obtained at grid point location (38,22). Above graph illustrates that, the deformation of around 380mm occurs at grid point (38, 22) on linear distribution of material as 80,60,40 percentage fine fraction which is 8m below surface. The deformation reduces to around 125mm at 50, 40, 35 and 30, 25, 20 linear distribution of percentage fine fraction at same location. At linear distribution of 20 percentage fine fraction alluvial deposits in 30m depth, maximum deformation is found around 23mm.

4.4. Deformation Histories Plot Vs Dam Height at Varying Percentage Fine Fraction

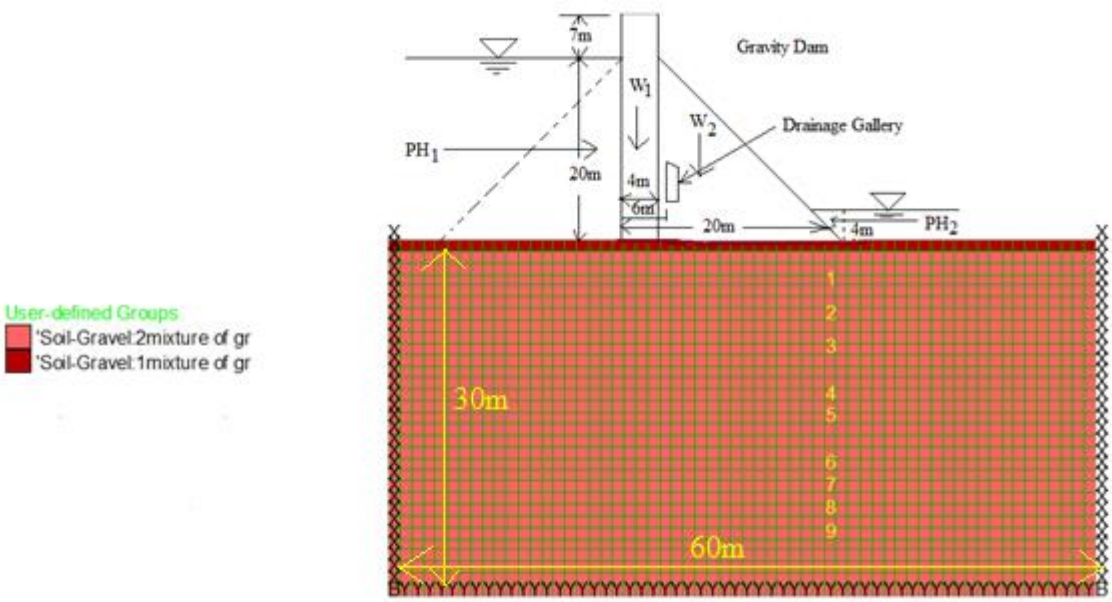

Figure 4.33: Model of Gravity Dam at 40 Percentage Fine Fraction

4.4.1. Deformation histories plot of 27m, 20m and 10m height dam at 40 Percentage Fine Fraction

The alluvial deposit is modelled at linear distribution of 40 percentage fine fraction. The dam is modelled at plain-strain condition using Mohr-Coulomb failure criteria. The depth and width of foundation material is 30m and 60m. The dam height is modelled for 27m, 20m and 10m height in finite difference method software. The maximum deformation occurs below the toe of dam at full reservoir conditions. The deformation subsequently decreases with increase in foundation depth. The maximum deformation histories of dam are plotted below the toe for three different dam height. The 27m height dam settles more than 80mm whereas 20m and 10m height dam settles less than 40mm.

Therefore, above graph illustrates that dam up to 20m height performed well for the 40-percentage fine fraction condition. But there is an excessive deformation for dam height of 27m.

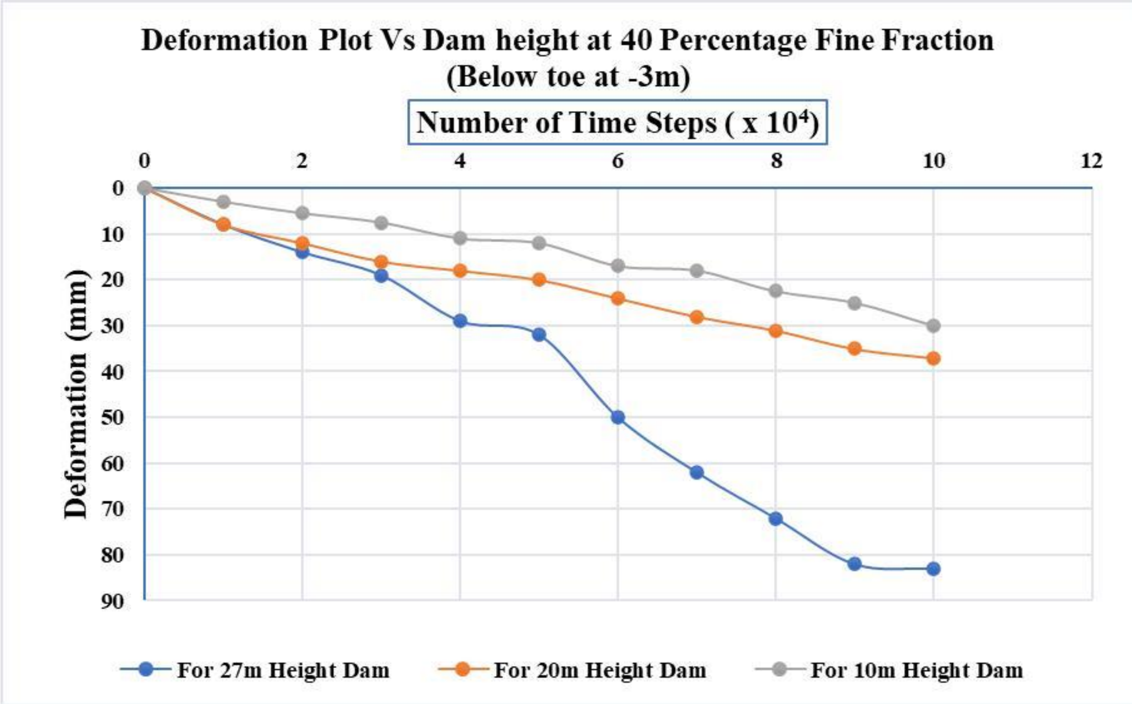

Figure 4. 34: Maximum Deformation on Alluvial deposits for varying dam height just below the toe at a depth of 3m from surface.

4.4.2. Deformation Plot Vs Dam Height for 30 Percentage Fine Fraction

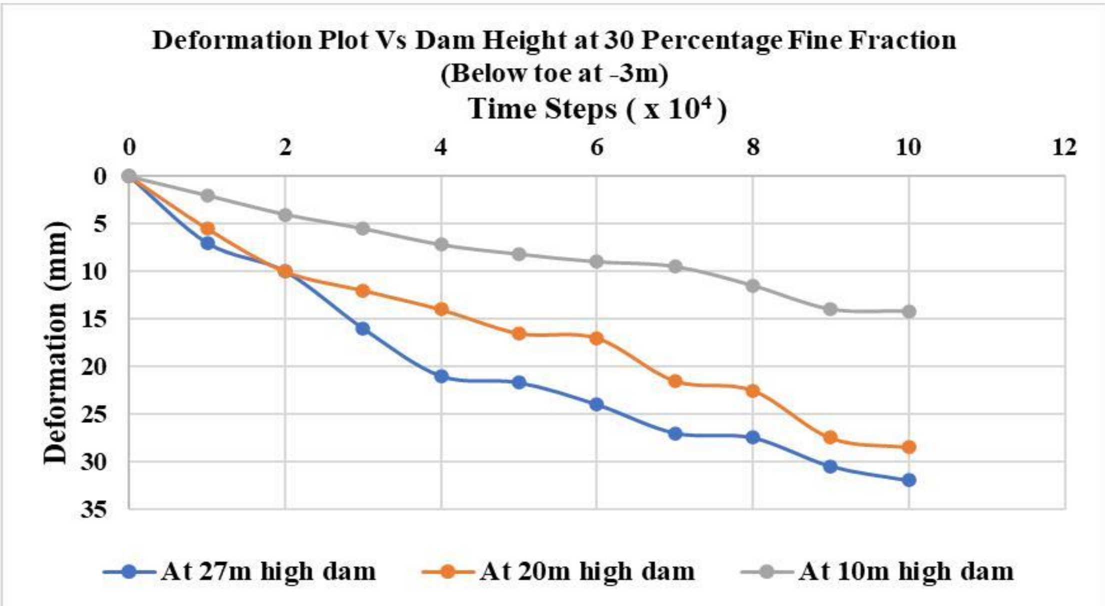

Figure 4. 35: Deformation on alluvial deposits due to varying Dam height at 30 percentage fine fraction

The 27m, 20m and 10m height dam are modelled at 30 percentage fine fraction. With all other boundary conditions same as stated in 40 percentage fine fraction. The maximum deformation occurs at 27m height dam which is around 33mm. And the deformation at 27m height dam and 10m height dam is found around 28mm and 14mm.

4.5. Maximum Excess Porewater Pressure Vs Dam Height at varying percentage Fine Fraction

4.5.1. Maximum excess pore pressure vs dam height at 40 percentage fine fraction

In all cases, the maximum excess porewater pressure is found just below the toe of dam. Excess porewater pressure is higher in 27m high dam its value subsequently decreases in 20m and 10 m high dam. The below graph illustrates that the excess porewater pressure decreases subsequently with the increase in depth. The maximum excess pore pressure developed at 27m height dam is found around 2.5 x 10^5 $\frac{\mathrm{N}}{\mathrm{m}^2}$. Similarly, the maximum excess pore pressure developed at 20m and 10 height dams are 1.3 x 10^5 $\frac{\mathrm{N}}{\mathrm{m}^2}$ and 0.5 x 10^5 $\frac{\mathrm{N}}{\mathrm{m}^2}$.

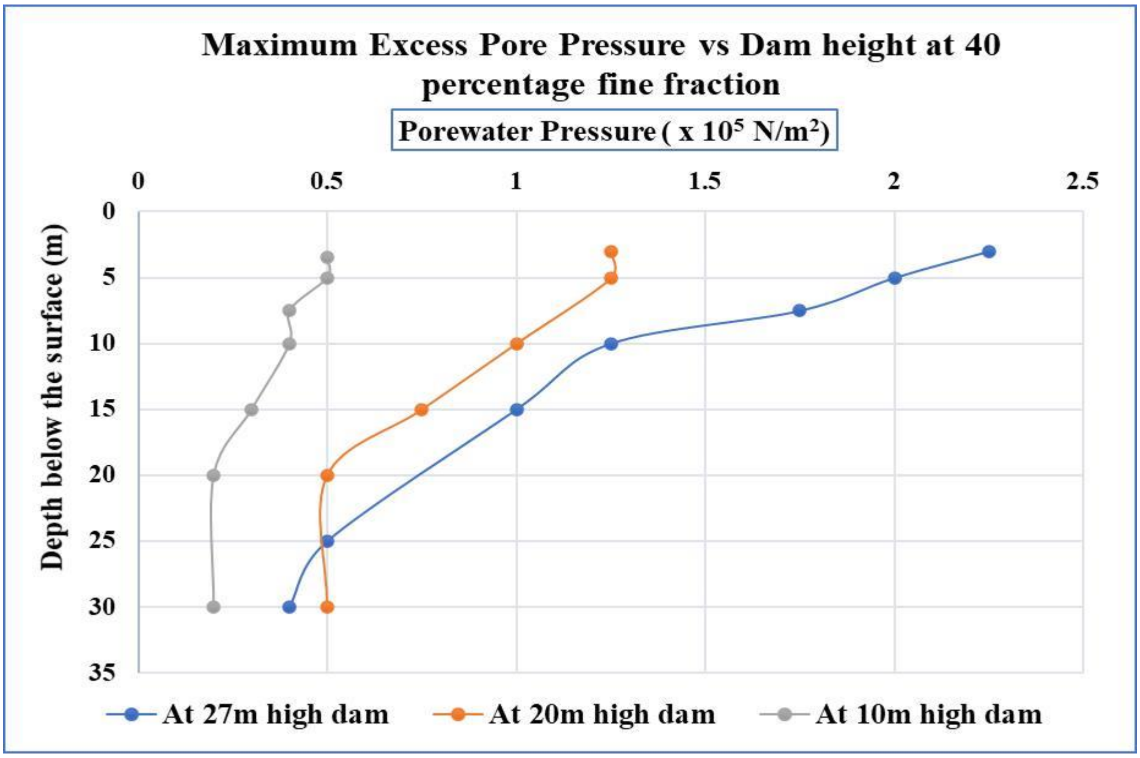

Figure 4. 36: Maximum Excess Porewater Pressure on Alluvial deposits for varying dam height.

4.5.2. Maximum excess pore pressure vs dam height at 30 percentage fine fraction

With decrease in percentage fine fraction the maximum excess pore water developed in 20m and 10m height dam increases whereas the maximum excess pore pressure decreases slightly in case of 27m height dam. The value of developed excess pore pressure for different dam height at 30 percentage fine fraction is illustrated as shown in figure below.

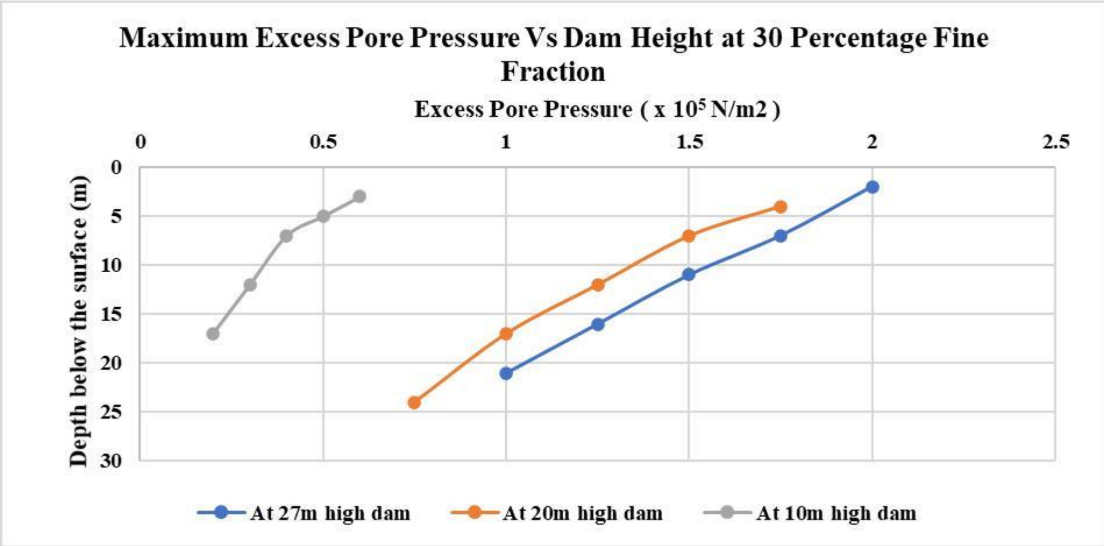

Figure 4. 37: Maximum Excess Pore Pressure developed on alluvial deposits due to varying Dam Height at 30 Percentage Fine Fraction

4.6 Modelling of Gravity Dam on Heterogenous Material Distribution

4.6.1 At 80 Percentage Fine Fraction

At 80 percentage fine fraction, the thickness of plate on which dam stands increases to 2m. The size of dam and foundation cross-section is same as above.

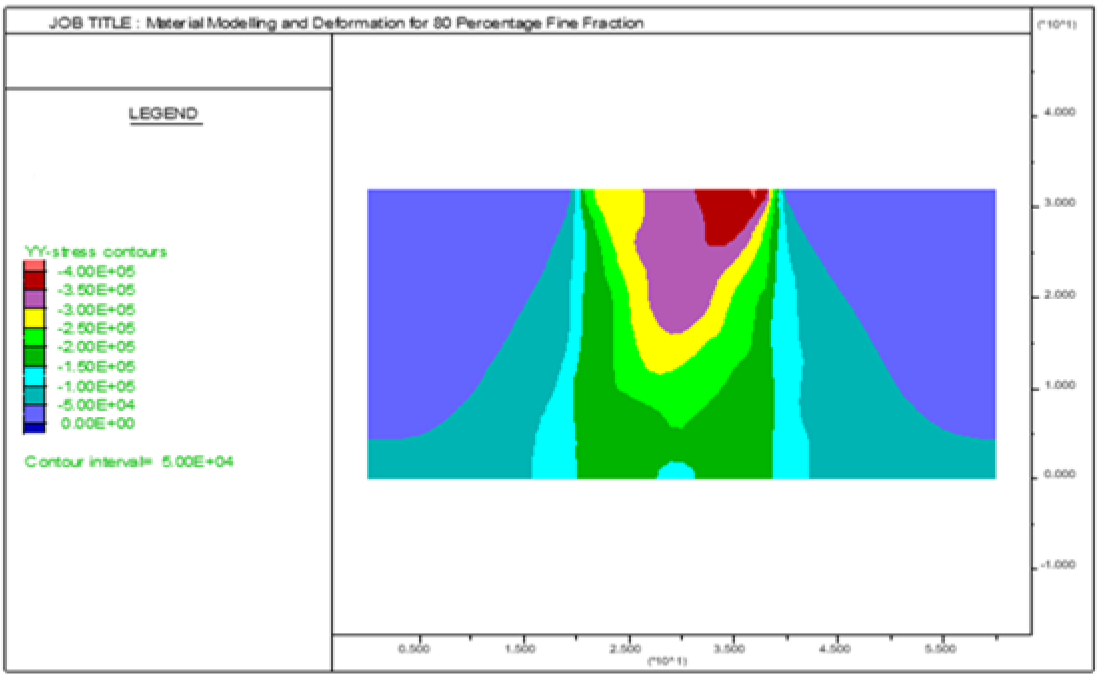

Figure 4. 38: Vertical Stress Contours of Dam at 80 Percentage Fine Fraction

The maximum vertical stress is found 3.5 X $10^5 \frac{N}{m^2}$ below the toe of foundation. Here, the vertical stress of dam foundation decreases with increase in depth. Due to excessive stress and displacement at this condition, dam settle more. Those models are not represented in this.

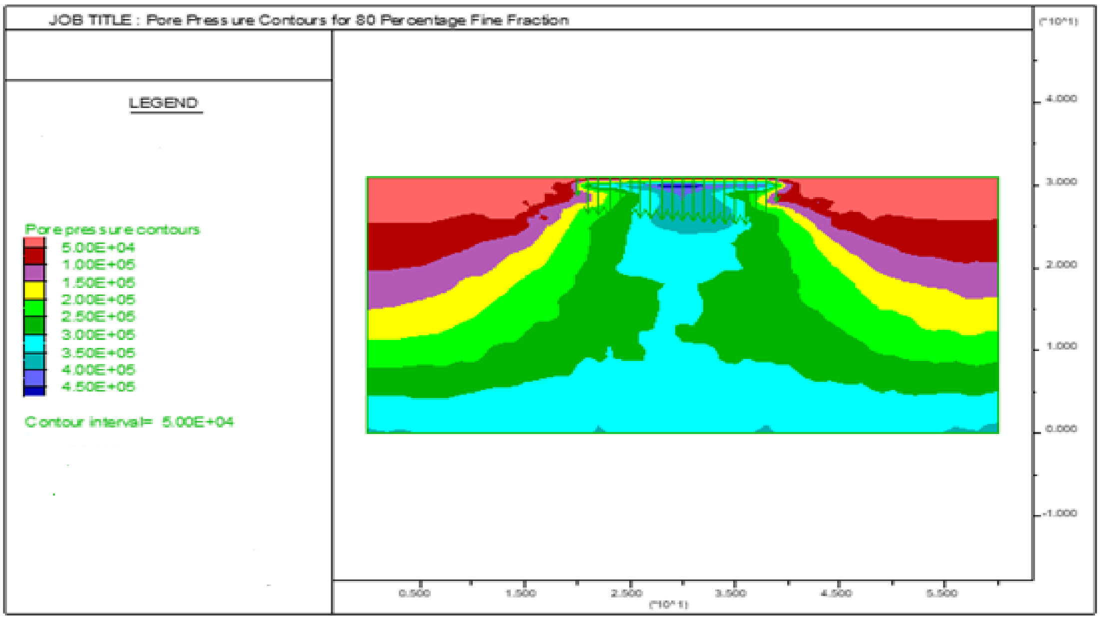

Figure 4. 39: Pore Pressure Contours of Dam at 80 Percentage fine fraction

The excess pore pressure occurs maximum at the top of dam foundation. The maximum pore pressure is found to be 4 X $10^5 \frac{N}{m^2}$ just below the toe portion of dam foundation. With increase in coarse content of foundation material the value of pore pressure decreases. The value of excess pore pressure decreases with decrease in depth as shown in above figure 4.35.

4.6.2 Material Modelling and deformation Calculation at 35 Percentage Fine Fraction

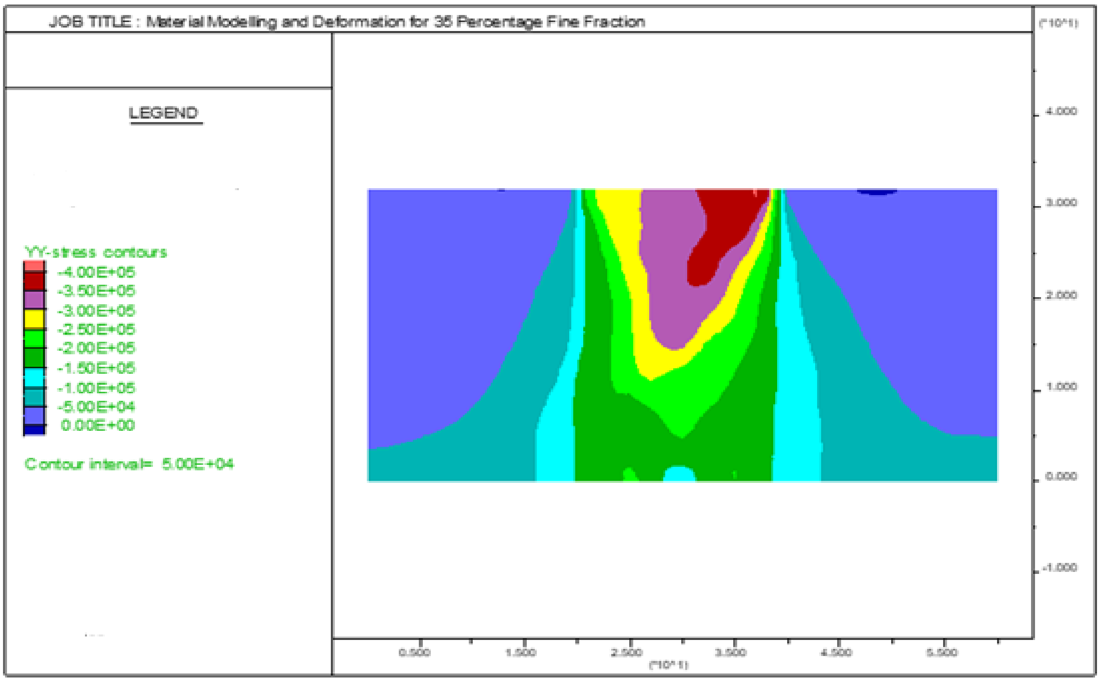

Figure 4. 40: Vertical Stress Contours at 35 Percentage fine Fraction

In all cases the vertical stress remains almost same. Combination of grouting doesn't change the vertical stress developed below the dam.

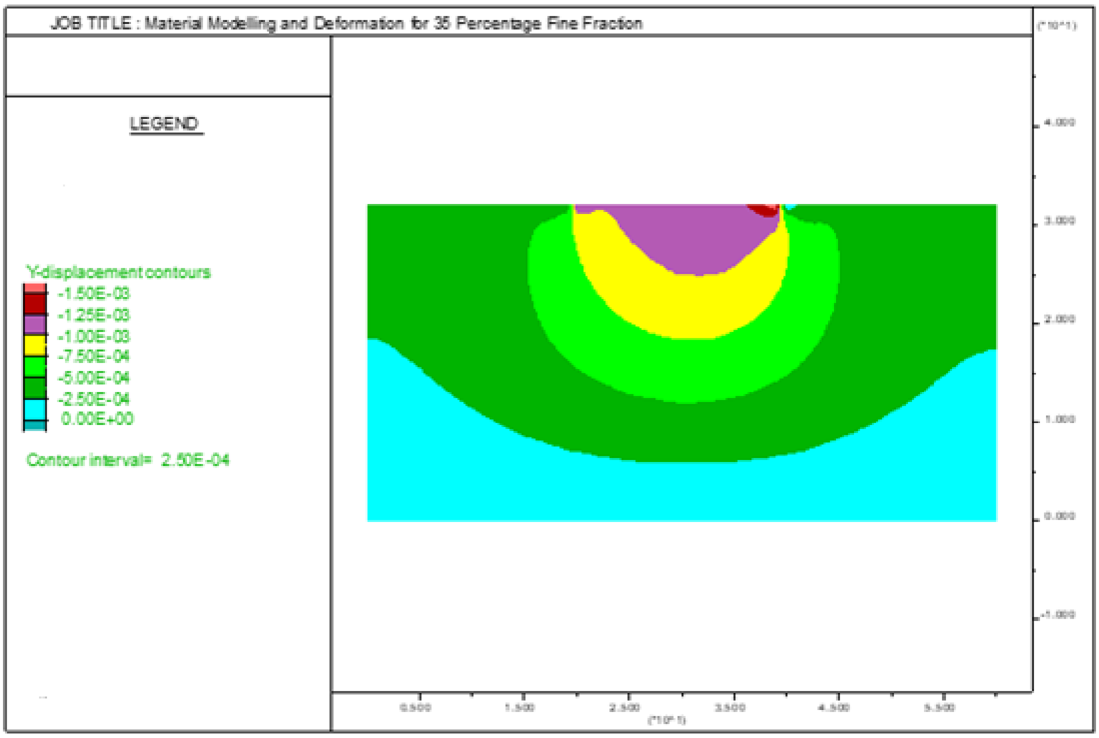

Figure 4. 41: Vertical deformation of dam at 35 percentage fine fraction

At 35 percentage fine fraction the maximum displacement below dam is found to be 1mm. The value of displacement decreases with respect to increase in depth.

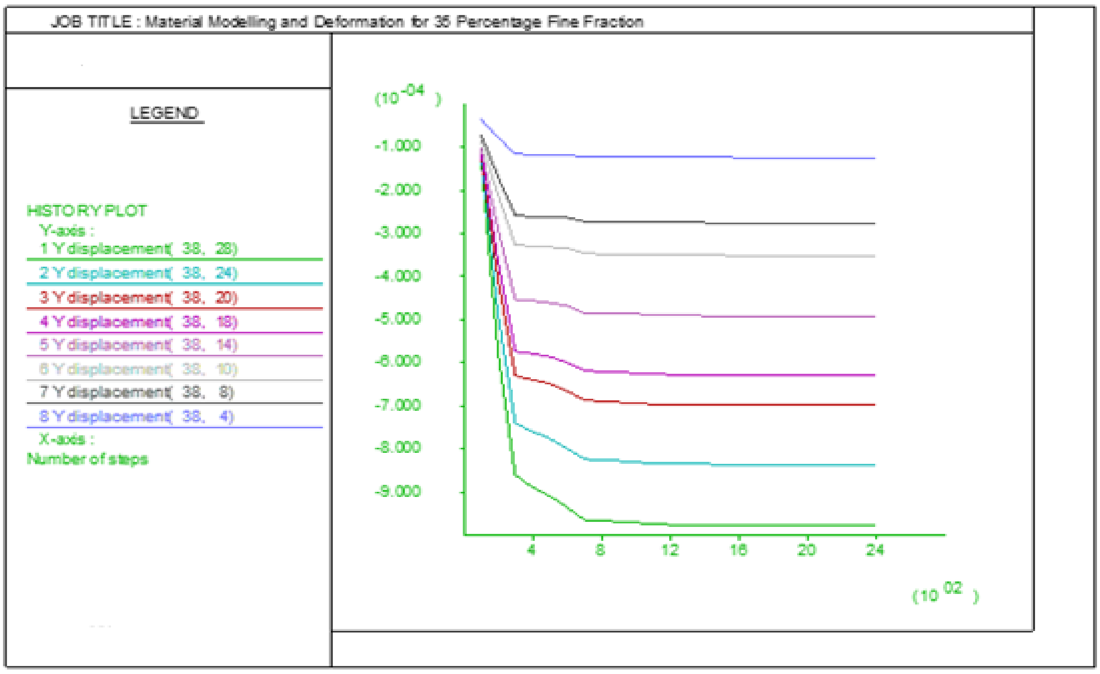

Figure 4. 42: Vertical deformation histories of dam at 35 percentage Fine fraction

The deformation in the subbase of dam is found to be comparatively more just below the toe due to more vertical stress at toe. The deformation histories at 8 different points below the toe of dam is found out in above figure 4.39. The maximum deformation is found to be around 9mm just below the toe of dam. The deformation histories at different time steps shows that the deformation decreases with respect increasing the depth.

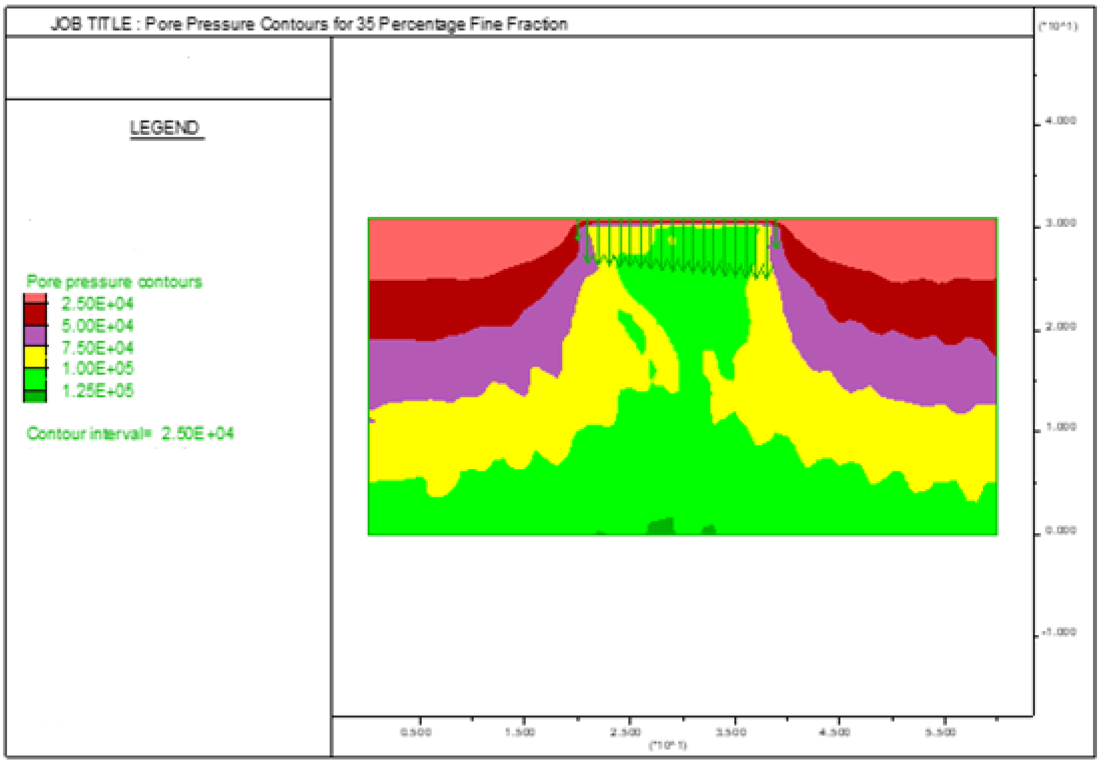

Figure 4.43: Pore Pressure Contours of foundation material at 35 percentage fine fraction

The excess pore pressure occurs maximum at the top of dam foundation. The maximum value of pore pressure in dam foundation at 35 percentage fine fraction is found to be $1.25 \text{ X } 10^5 \frac{\text{N}}{\text{m}^2}$. It was four times less than pore pressure at 80 percentage fine fraction. With increase in coarse content of foundation material the value of pore pressure decreases. The value of excess pore pressure decreases with decrease in depth is illustrated in above figure 4.40.

4.6.3 At 20 Percentage Fine Fraction

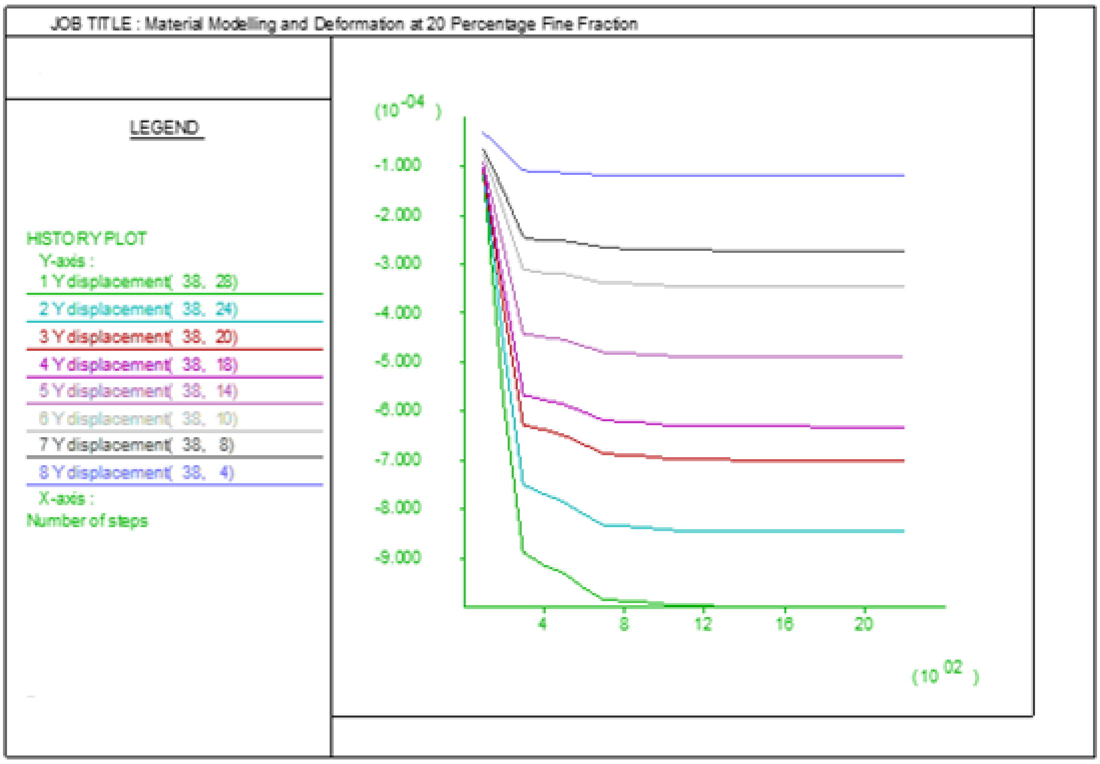

Figure 4. 44: Deformation histories of dam at 8 different points

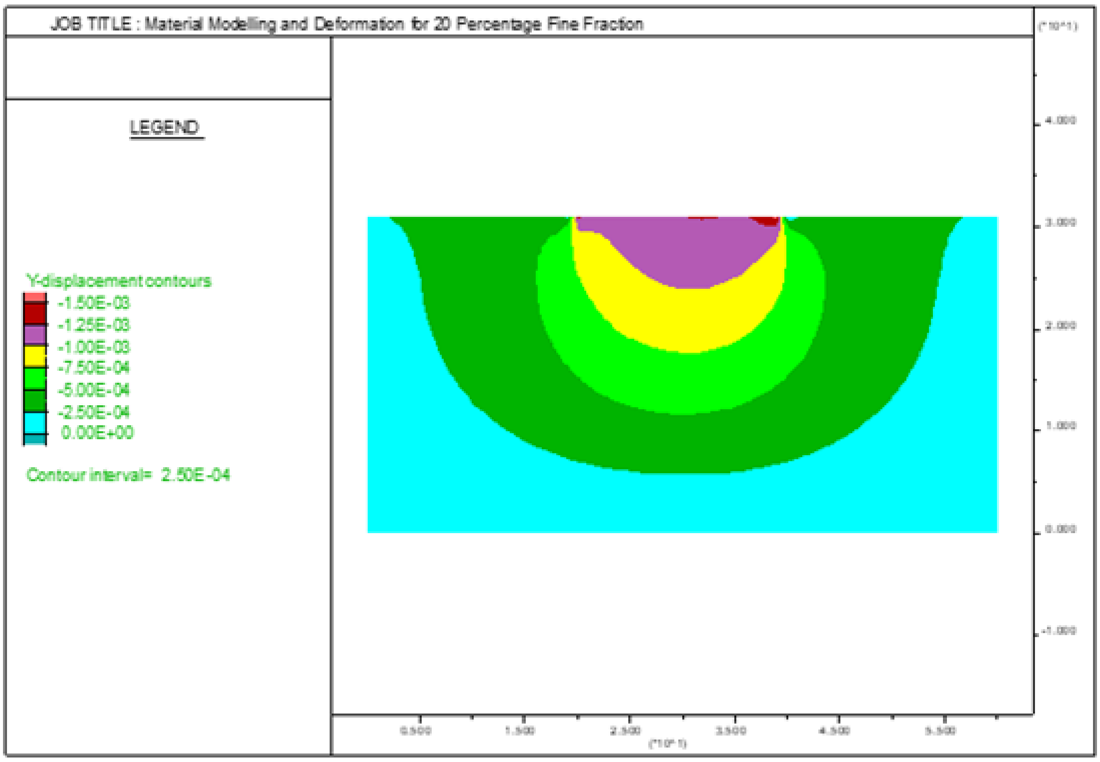

Figure 4. 45: Vertical Displacement contours at 20 Percentage fine fractions

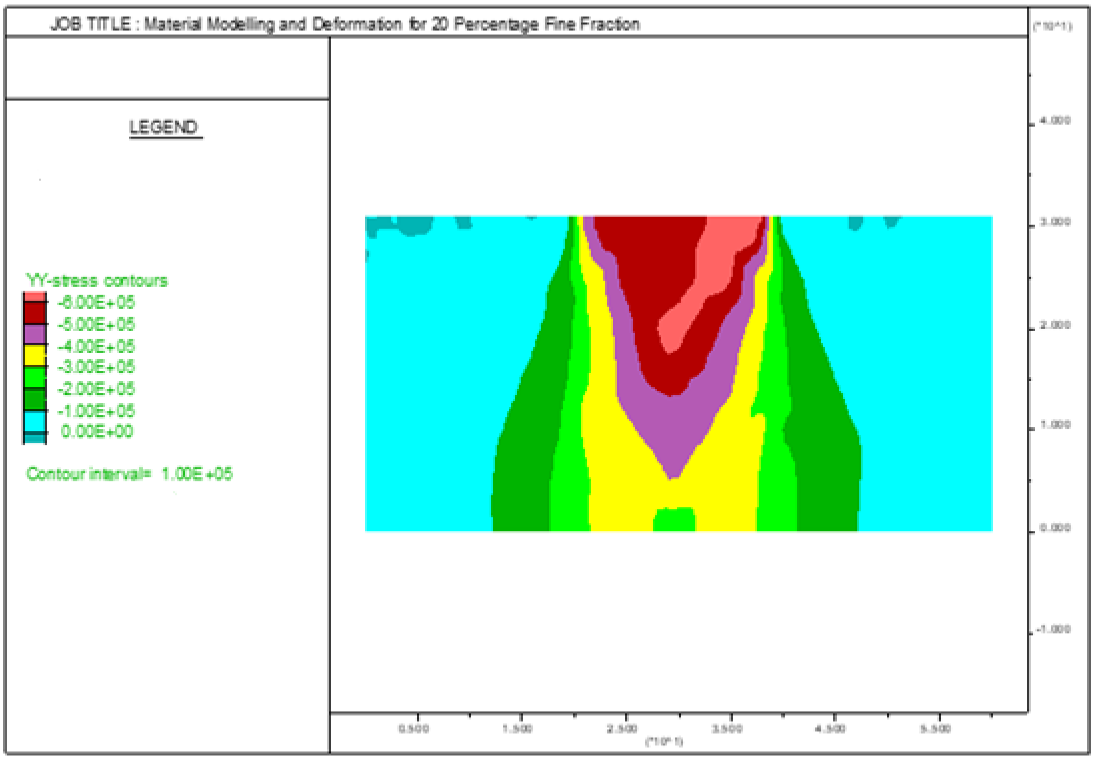

Figure 4. 46: Vertical Stress Contours at 20 percentage fine fraction

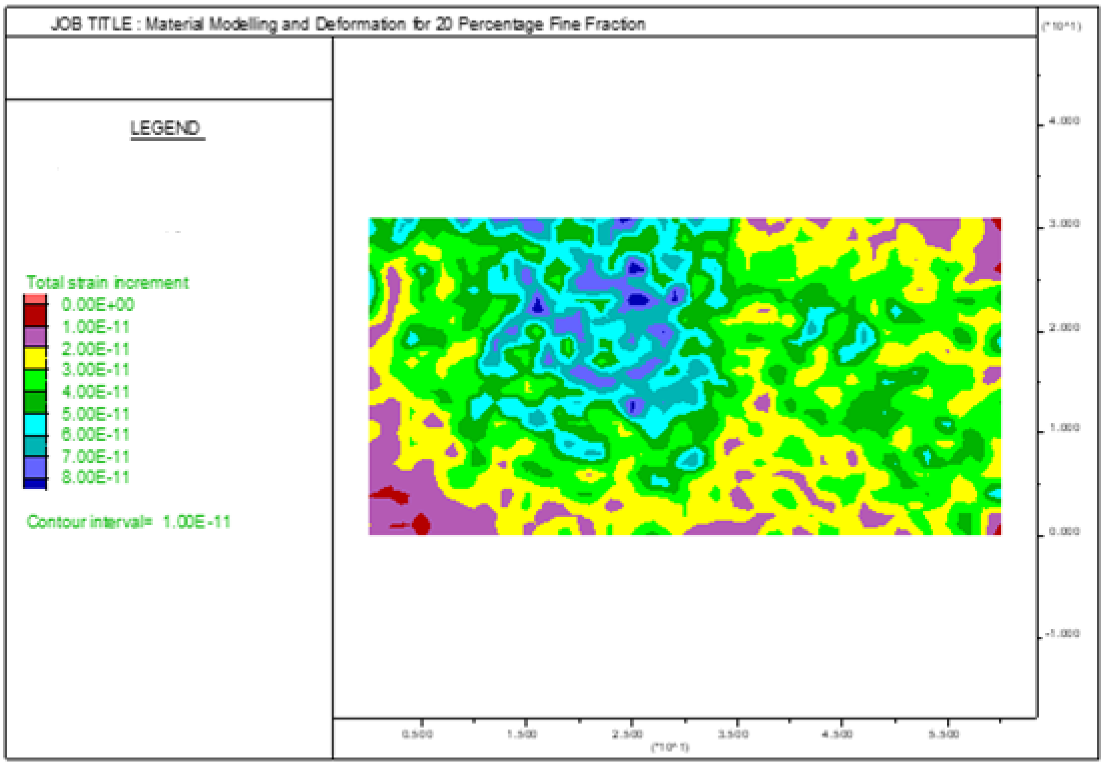

Figure 4. 47: Total strain increment at 20 percentage fine fraction

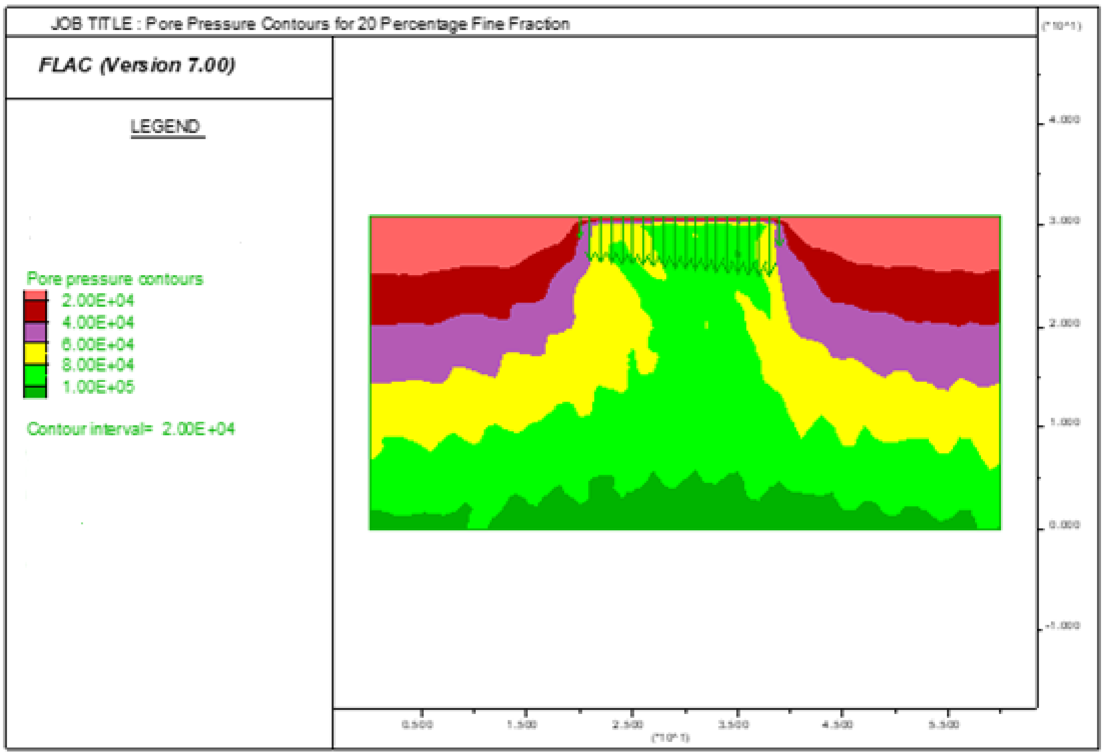

Figure 4. 48: Pore Pressure Contours of foundation material at 20 percentage fine fraction

The excess pore pressure occurs maximum at the top of dam foundation. The maximum value of pore pressure in dam foundation at 20 percentage fine fractions is 1 X $10^5 \frac{N}{m^2}$. It was nearly four times less than pore pressure at 80 percentage fine fraction. With increase in coarse content of foundation material the value of pore pressure decreases. The value of excess pore pressure decreases with decrease in depth is illustrated in above figure 4.46.

4.7 Seepage Analysis Results with and without cutoff wall for decreasing fine fraction

4.7.1 Seepage Analysis

When fine fraction of alluvial deposits is consolidated by grouting, there is decrease in void content. It makes the soil less permeable which ultimately decreases the seepage as shown in below figure 4.47, 4.48, 4.49, 4.50, and 4.51. As illustrated in those figures, value of seepage decreases from 0.1071 m^3/s in natural condition to 0.0000990m^3/s at 80 percentage decrease in porosity. From these results, it is clear that decrease in fine fraction of alluvial deposits decreases the quantity of seepage. Also, with the provision of cutoff wall of 8m length the length of seepage path increases which ultimately decreases the value of seepage as illustrated in fig: 4.51 to fig: 4.55. With same material property and dam load the value of total seepage decreases due to cutoff wall of 8m depth.

Table 4. 4: Summary of Seepage Analysis

S.N.	Void ratio(e)	Porosity(n)	Permeability (k) (m/sec)	Seepage Without cutoff wall (GeoStudio) (m^3/sec)	Seepage (8m Cutoff wall) (GeoStudio) (m^3/sec)
1.	2.22	0.69	0.0134	0.10701	0.08948
2.	1.23	0.552	0.000294	0.090242	0.00196
3.	0.70	0.414	0.000725	0.0057899	0.00484
4.	0.38	0.276	0.000140	0.001181	0.000934
5.	0.16	0.138	0.0000124	0.0000990	0.00008281

4.7.2 Seepage analysis without cutoff wall

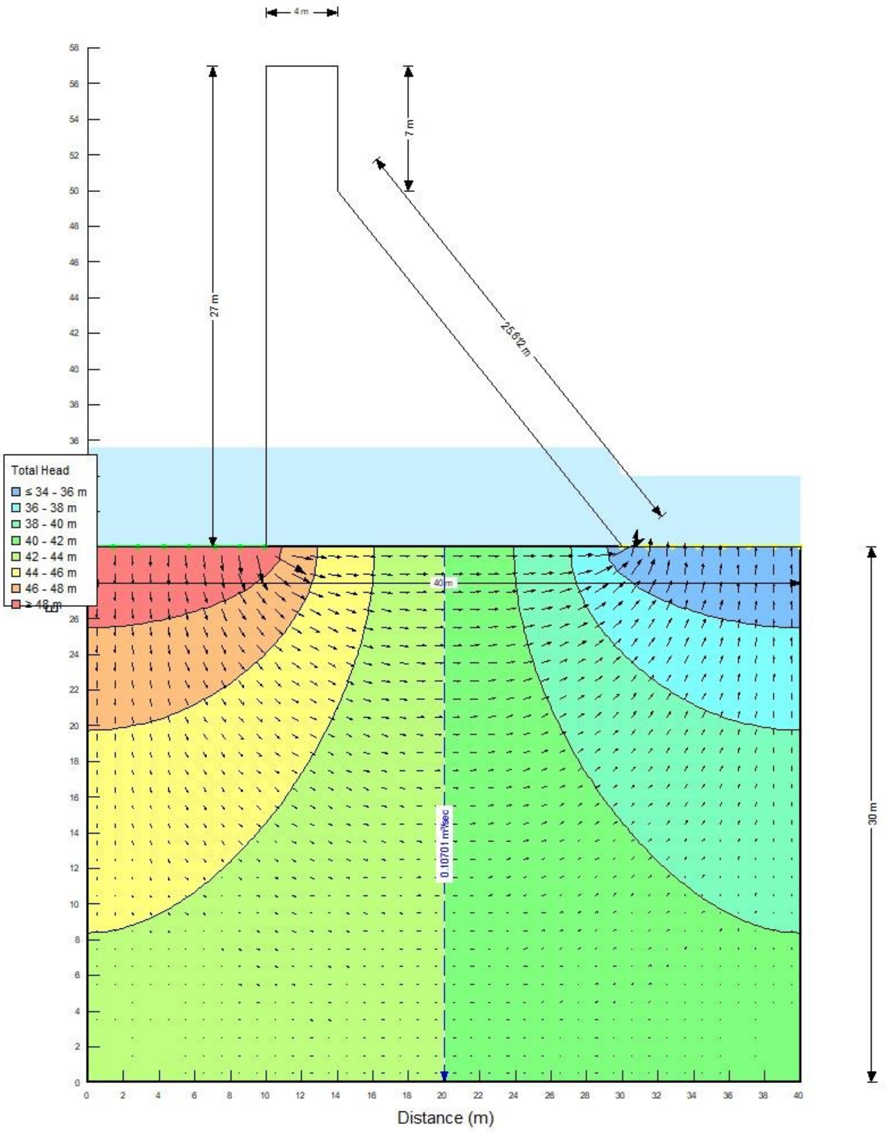

Figure 4. 49: Seepage Analysis at Natural State

A seepage analysis is performed in GeoStudio at natural condition with modelling of 27m high gravity dam. The upstream level of water is considered as 20m whereas downstream level of water is 4m. The value of permeability, volumetric compressibility and volumetric water content is put in model as described in methodology chapter. At natural state the value of seepage is found to be 0.10701 m^3/sec. Seepage is high at natural state due to excessive amount of percentage fine fraction.

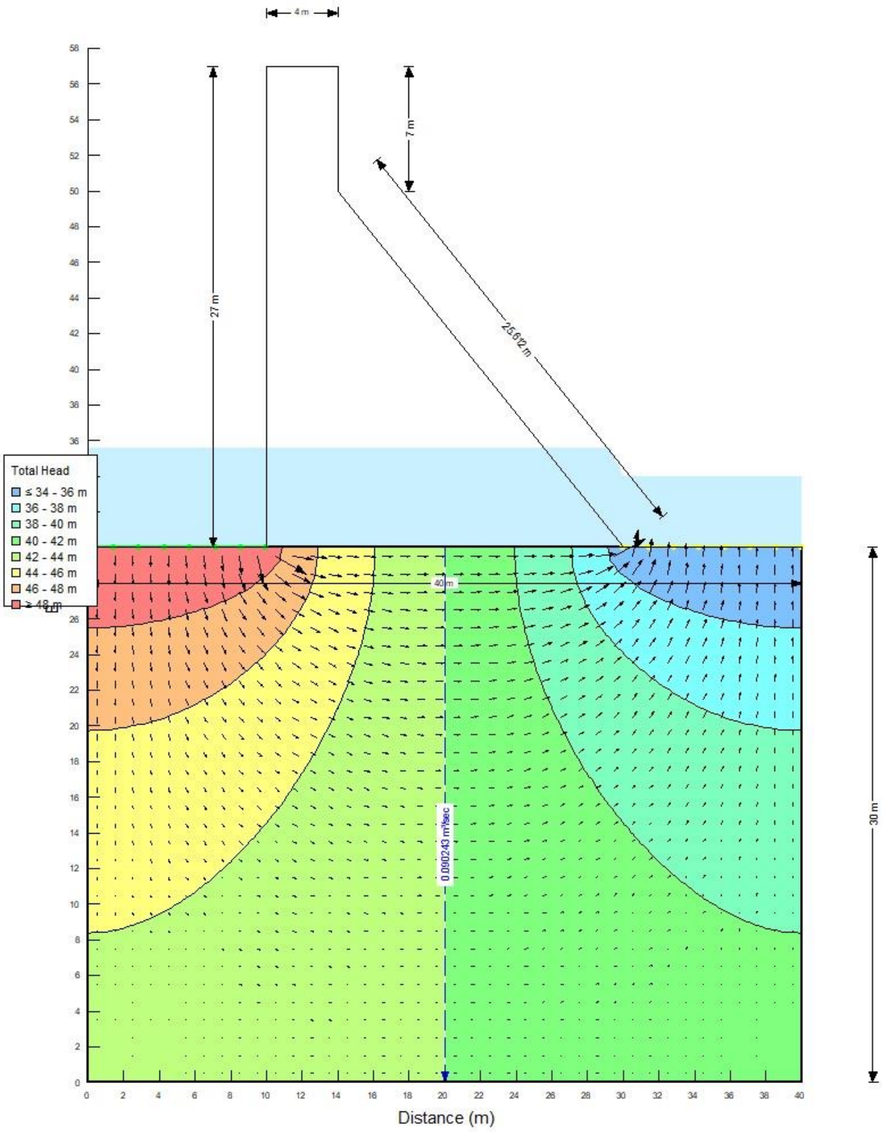

Figure 4. 50: Seepage analysis at decrease in 20 percentage porosity

Then compaction and consolidation grouting are done in dam foundation to increase the stiffness of foundation material. With decrease in 20 percentage porosity, the value of seepage is found to be 0.090242 m^3/sec. It shows that increase in grouting, increases the stiffness of subbase material which decreases the porosity and finally decreases the seepage.

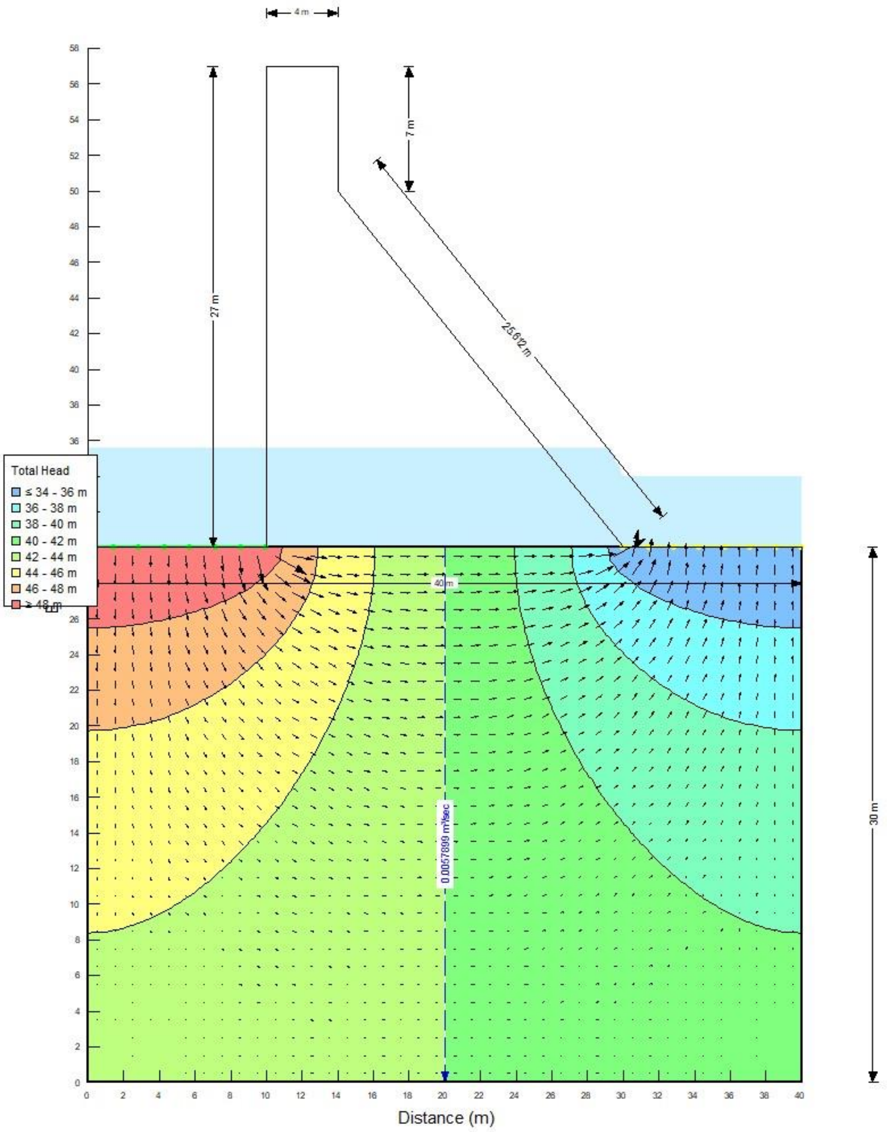

Figure 4. 51: Seepage analysis at Decrease in 40 percentage porosity

The value of seepage is found to be 0.0057899 m^3/sec with decrease in 40 percentage porosity.

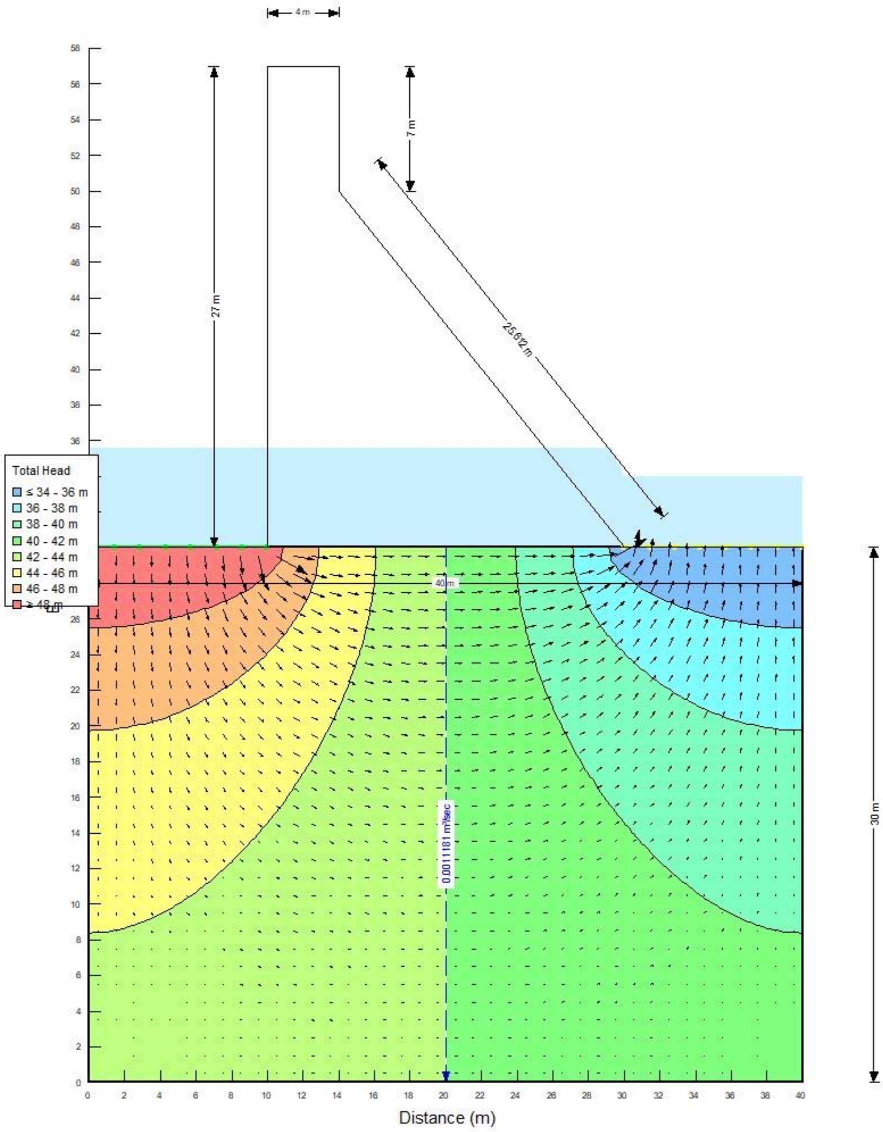

Figure 4. 52: Seepage analysis at decrease in 60 percentage porosity

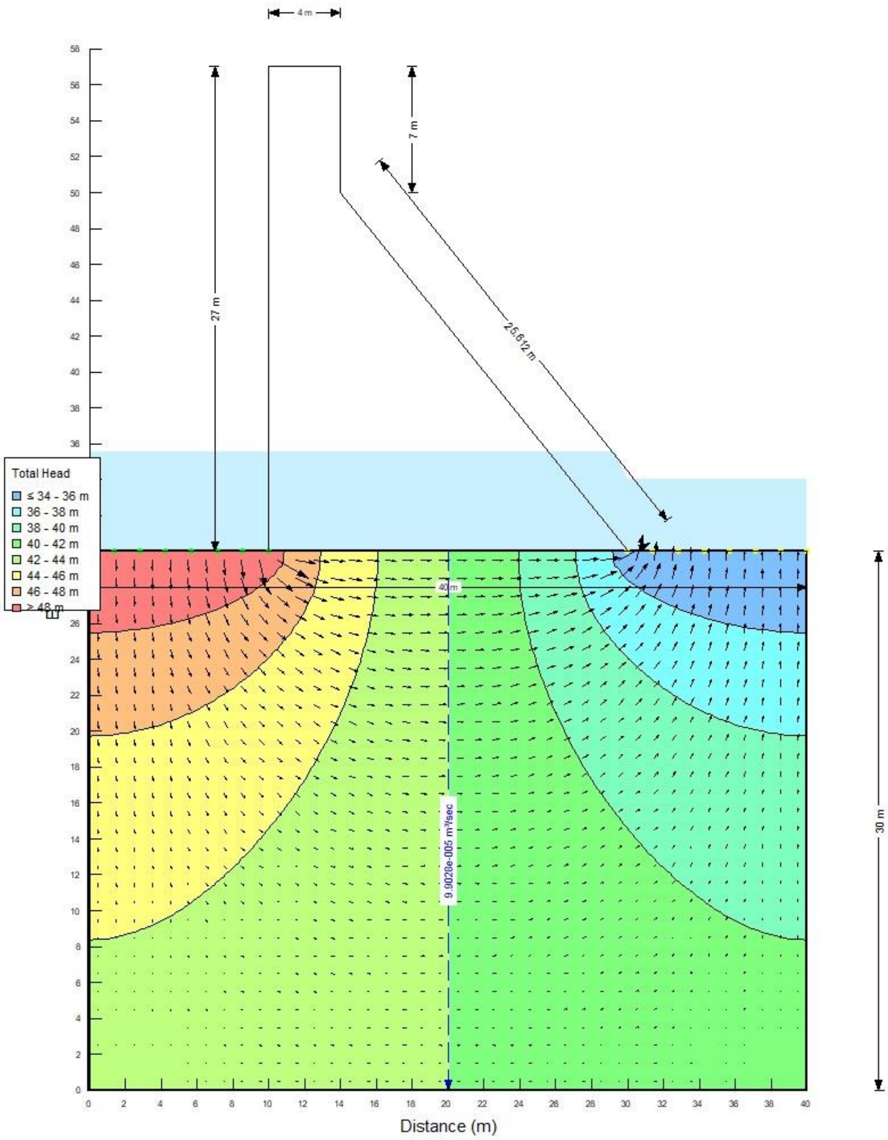

Figure 4. 53: Seepage analysis at decrease in 80 percentage porosity

4.7.3 Seepage analysis with 8m cutoff wall

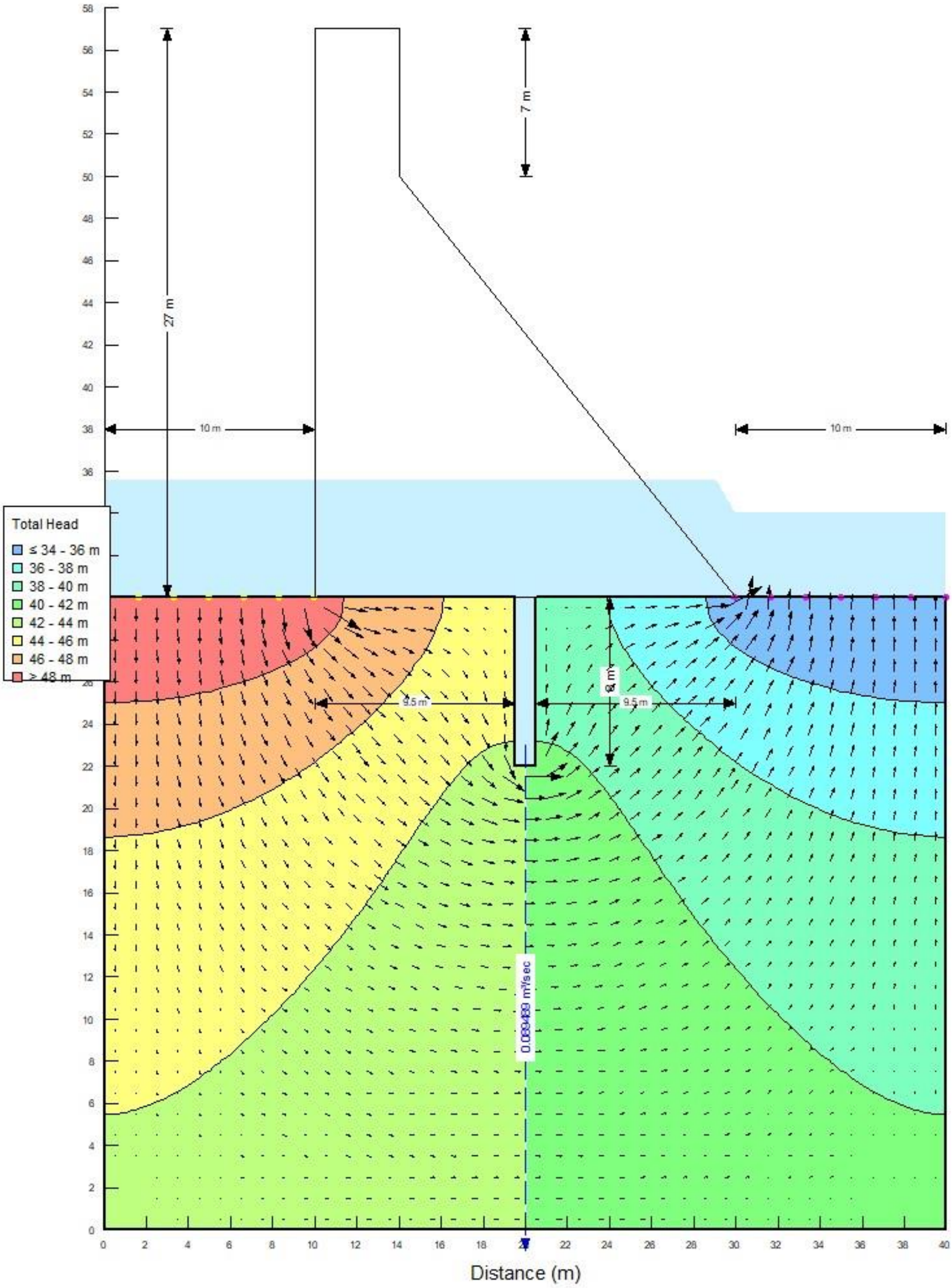

Figure 4.54: Seepage analysis at natural state with cutoff wall

The value of seepage is found to be 0.08948 m^3/s at 80 percentage decrease in porosity.

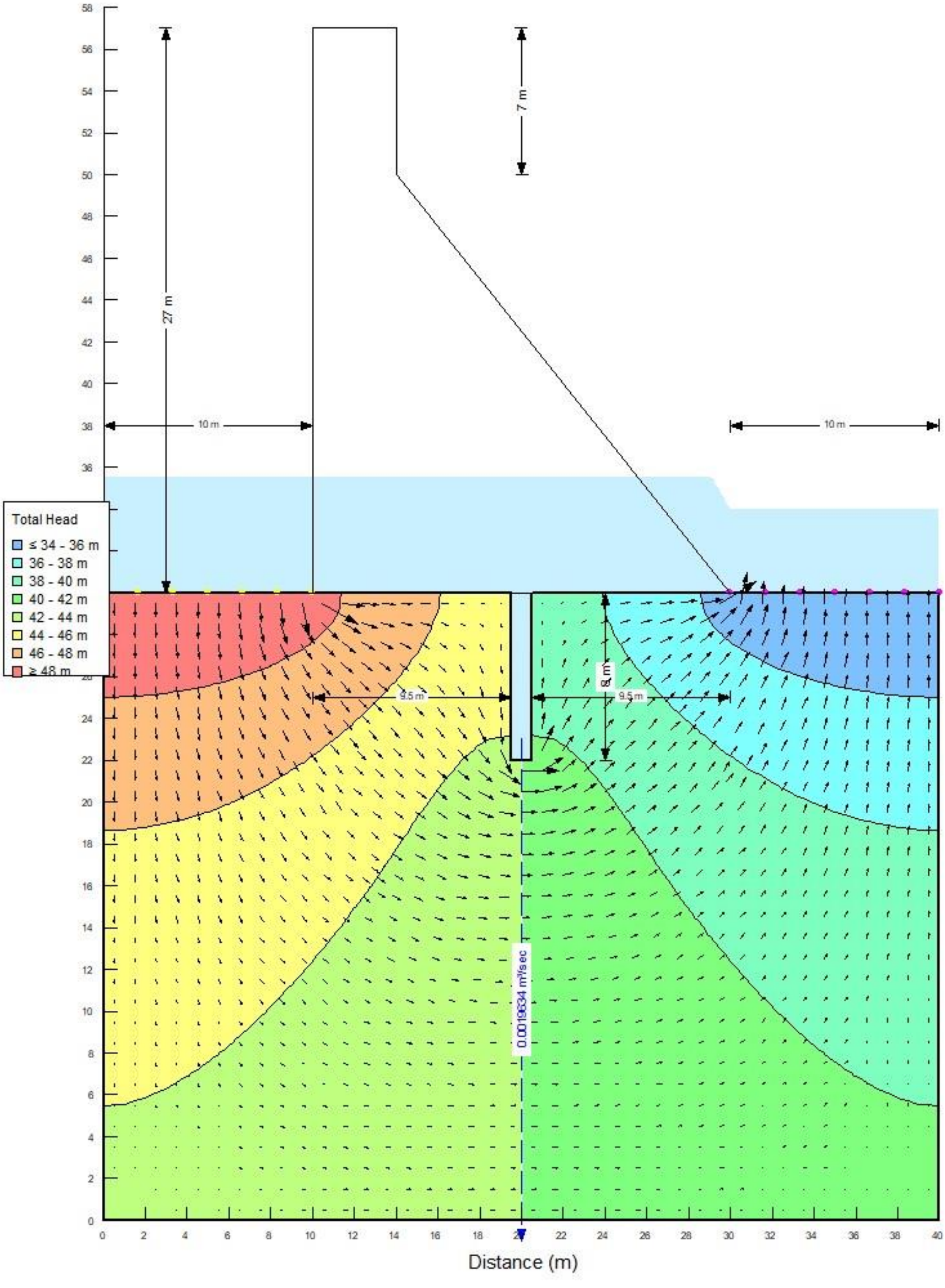

Figure 4.55: Seepage analysis at decrease in 20 percentage porosity

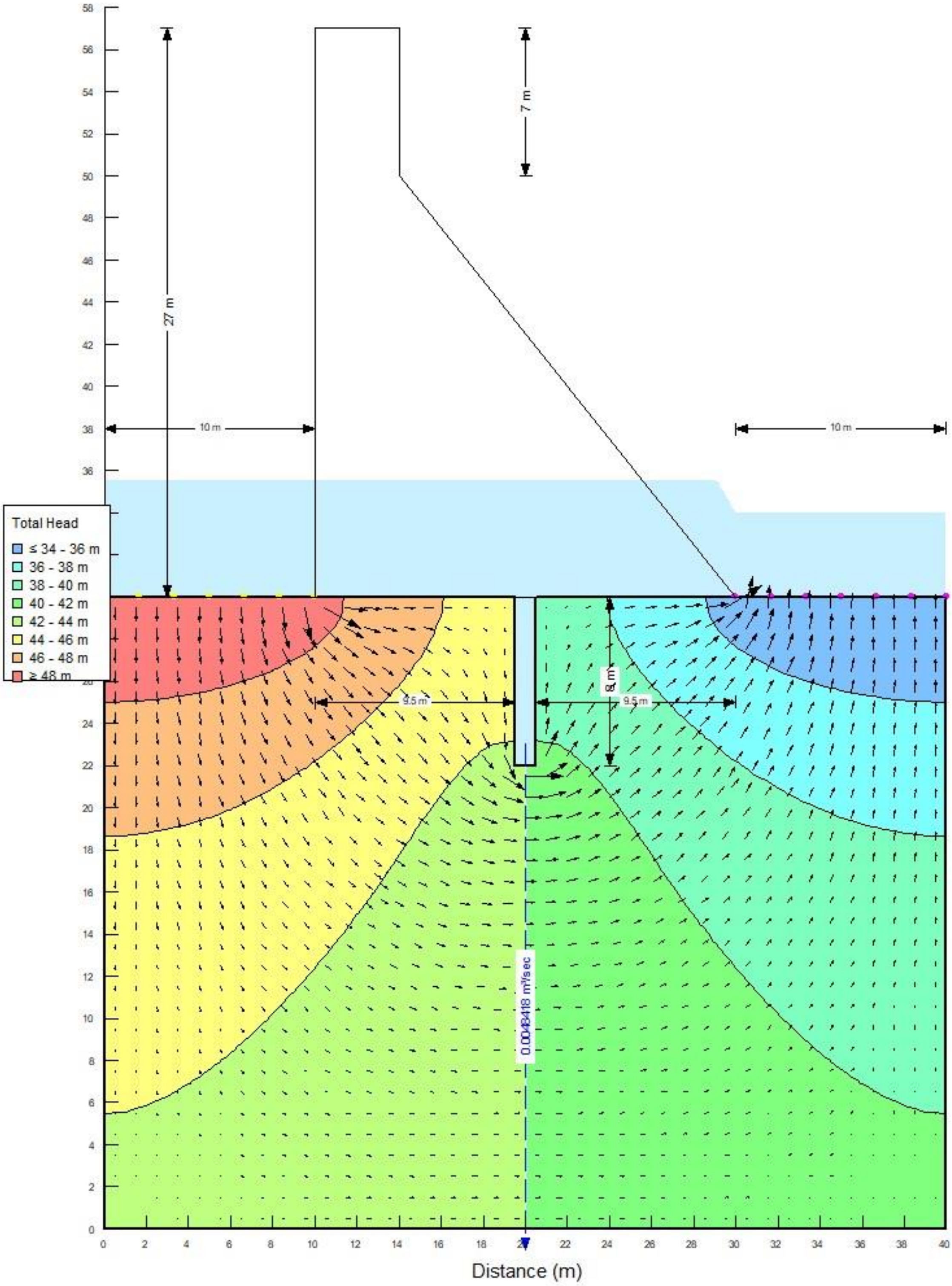

Figure 4. 56: Seepage analysis at decrease in 40 percentage porosity

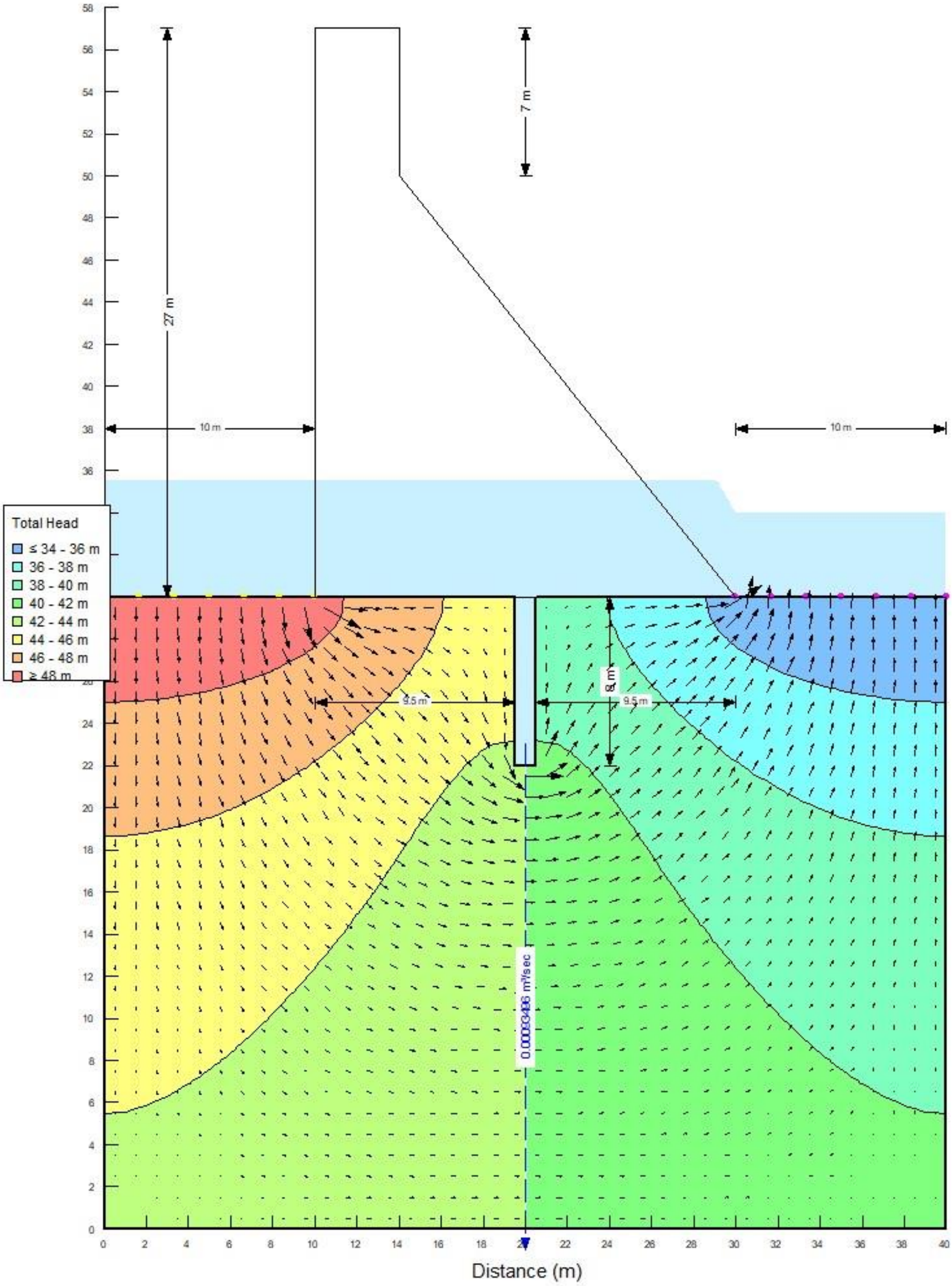

Figure 4.57: Seepage analysis at decrease in 60 Percentage porosity

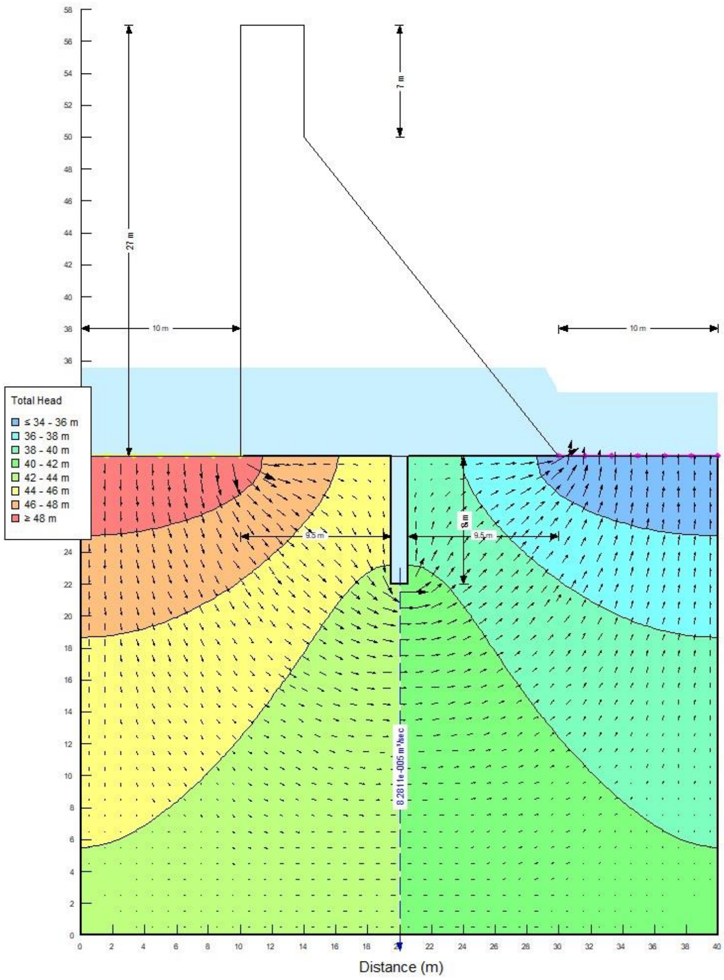

Figure 4.58: Seepage analysis at decrease in 80 percentage porosity

The value of seepage is found to be 0.00008281 m^3/s after 80 percentage decrease in porosity of alluvial deposits in the presence of 10m cutoff wall.

In sum up, the value of seepage decreases with decrease in porosity. When grouting is done in alluvial foundation, the void ratio of fine grain decreases. With decrease in void ratio, the permeability of soil decreases. With decrease in permeability, seepage below the dam decreases. The value of seepage decreases with the provision of cutoff walls.

5. CONCLUSIONS

This study presented a methodology to analyse the stability of dam foundation in alluvial deposit in natural and improved soil strata by means of analytical and numerical method. First a relationship is established between percentage fine fraction with moduli of elasticity and density from the analysis of MASW survey data and borehole data on alluvial deposits. The relationship shows that mechanical properties of soil increase with decrease in percentage fine fraction. This relationship is used to find the mechanical properties of soil having different fine fractions of alluvial deposits. The results are used to model the dam foundation in alluvial soil deposit.

Models representing RCC dams on top of the alluvial soil deposits are prepared. The alluvial soil is modelled at natural and improved condition. The improvement is from the reduction of percentage fine fraction by means of grouting i.e. the cementation of fine material. The model simulated the process and the result obtained showed the decrease in deformation, pore water pressure and seepage with the decrease in fine fraction. In total 14 numbers of models are analysed in FDM to represent the reduction in fine fraction from (0-75) % which calculate the value of vertical stress, deformation and porewater pressure. Ten models are analysed in GeoStudio which represented the reduction in fine fraction with and without provision of cutoff walls which gives the value of seepage. The result from the numerical model representing (10-80) % reduction in fine fraction clearly depicts the performance of dam in alluvial foundation will be better with the replacement of the fine fraction voids by cemented material. For the 27m height gravity dam in assumed alluvial soil, the dam results of indicating settlement, pore pressure, vertical stress after reduction in 75% fine volume.

The numerical model considered the homogeneous and random distribution of the fines and coarse material. The homogeneous material assumed uniform value of strength parameters K, G and ρ, i.e., average value of boulders and fines. The heterogenous material model represented the random distribution of different fraction of boulders and fines and consequently the strength parameters. When an equal stress is applied at linear homogenous material model and random heterogenous material model. At linear distribution of material at 20 percentage fine fraction the maximum deformation is around 50mm whereas for heterogeneous material distribution maximum deformation is 1mm. This shows that bearing capacity of heterogenous material model is higher than homogenous material model at equivalent percentage fine fraction. The vertical stress on alluvial deposits is found higher in case of heterogenous material distribution than homogenous material distribution at equivalent percentage fine fraction.

The three dams of 27m, 20m and 10m high are modelled for 40 percentage fine fraction and 30 percentage fine fraction. Then, the value of deformation and excess pore pressure are plotted at different timesteps. The result shows that the deformation on alluvial deposits decreases with decrease in percentage fine fraction. And the deformation and excess porewater pressure decreases with increase in depth of deposits.

Seepage analysis result shows that seepage in alluvial deposits decrease with decrease percentage fine fraction of deposits. And, the provision of cutoff wall decreases the value of seepage by almost 10 times than without cutoff wall.

Moreover, the method presented to analyse the stability of dam foundation in alluvial foundation represented the load deformation mechanism in the foundation at the natural and improved soil. The numerical model result showed the validity of the method.

5. LIMITATION

- For more accurate results a greater number of MASW survey at different location having alluvial soil as predominant material is required with a greater number of borehole data.
- It doesn’t account for earthquake forces, silt pressure, wave pressure.

6. RECOMMENDATIONS

1. The percentage fine fraction relationship with moduli of elasticity and density for alluvial deposits find out in this thesis can be used for future case study. But it is recommended to collect many borehole data and MASW or microtremor survey data in alluvial deposits to represent actual field conditions. And established a relationship to find out mechanical parameters for alluvial deposits.

2. By doing combination of compaction and consolidation grouting and drilling the core in field we can found out the value of Bulk modulus of elasticity, shear modulus of elasticity and density for different degree of percentage fine fraction.

7. REFERENCES

1. Buczek, H. S. et al. (2018) ‘Clear Lake Dam Replacement: RCC Dam on a Challenging Soil Foundation’, Rocky Mountain Geo-Conference 2018 GPP 12, pp. 76–87.
2. Fell, R. et al. (2014) Geotechnical Engineering of Dams, 2nd Edition. 2nd edn, Geotechnical Engineering of Dams, 2nd Edition. 2nd edn. CRC Press/Balkema. doi: 10.1201/b17800.
3. Han Jie (2015) PRINCIPLES AND PRACTICE OF GROUND IMPROVEMENT.
4. Hedayati Talouki, H. et al. (2015) ‘Assessment and presentation of a treatment method to seepage problems of the alluvial foundation of Ghordanloo dam, NE Iran’, Journal of the Geological Society of India. doi: 10.1007/s12594-015-0227-2.
5. HOSSEIN, H. et al. (2015) ‘Assessment and Presentation of a Treatment Method to Seepage Problems of the Alluvial Foundation of Ghordanloo Dam , NE Iran’, 85(March), pp. 377–384.
6. Jafarzadeh, F. and Garakani, A. (2013) ‘Foundation Treatment of Embankment Dams With Combination of Consolidation and Compaction Groutings’.
7. Monley, G. et al. (2018) ‘Rocky Mountain Geo-Conference 2018 GPP 12 76’, Rocky Mountain Geo-Conference 2018, 12, pp. 76–87.
8. Pawson, J. R. and Russell, E. (2014) ‘Skhalta Dam - Design of a hardfill dam founded on deep alluvium and lacustrine deposits’, in Maintaining the Safety of Our Dams and Reservoirs - Proceedings of the 18th Biennial Conference of the British Dam Society. doi: 10.1680/mdam.60340.148.
9. Sharma, R. H. and Awal, R. (2013) ‘Hydropower development in Nepal’, Renewable and Sustainable Energy Reviews. Elsevier, 21, pp. 684–693. doi: 10.1016/j.rser.2013.01.013.
10. Stephenson, D. and Meadows, M. E. (1986) ‘Kinematic hydrology and modelling.’, Kinematic hydrology and modelling.
11. Uromeihy, A. and Barzegari, G. (2007) ‘Evaluation and treatment of seepage problems at Chapar-Abad Dam , Iran’, 91, pp. 219–228. doi: 10.1016/j.enggeo.2007.01.012.
12. Urumović, K. and Urumović, K. (2014) ‘The effective porosity and grain size

relations in permeability functions', Hydrology and Earth System Sciences Discussions, 11(6), pp. 6675–6714. doi: 10.5194/hessd-11-6675-2014.

13. Weaver, K. and Bruce, D. (2013) Dam Foundation Grouting, ASCE PRESS. doi: 10.1061/9780784407646.ch09.